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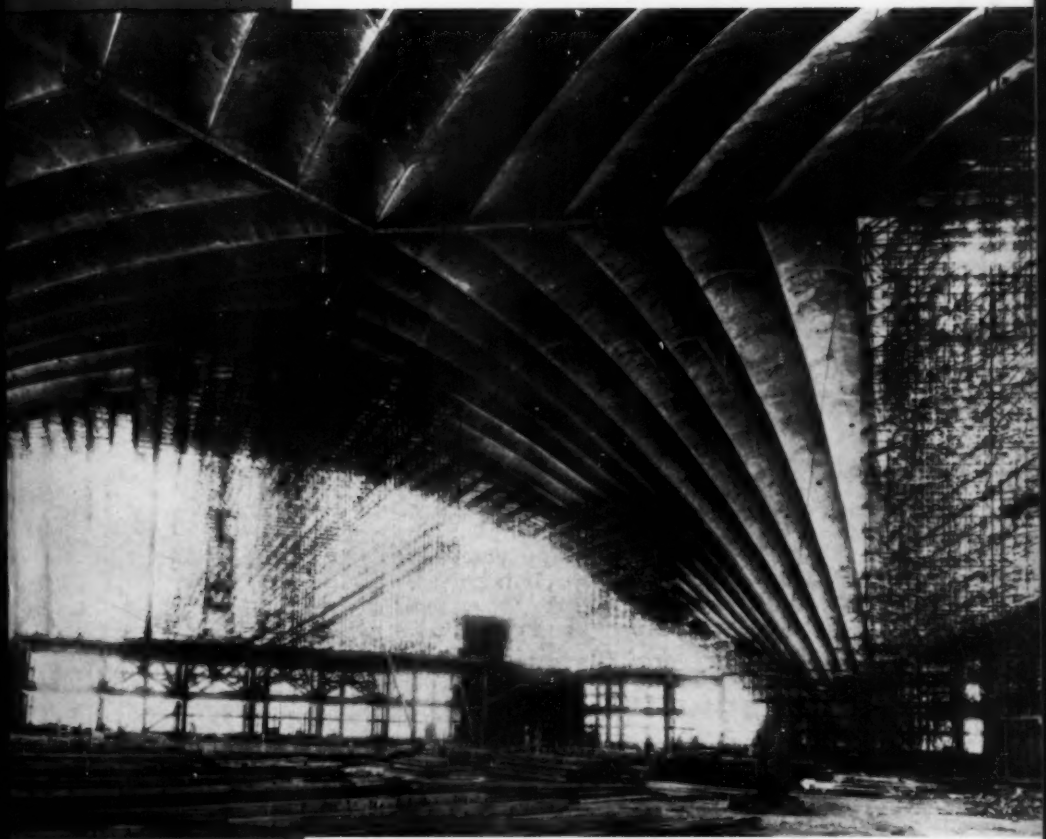

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ACI COMMITTEE 622 submits a report on "Formwork for Concrete," the result of more than 6 years of work by ten subcommittees. Prepared in three parts, the report discusses present practice, problems, and need for improved practice; the responsibility of the engineer or architect; and suggested requirements for formwork practice.

FRANCO LEVI reports on "The Work of the European Concrete Committee," summarizing the conclusions which have so far been approved by the Comité Européen du Béton.

ROBERT G. MATHEY and DAVID WATSTEIN present a résumé of their "Investigation of Bond in Beam and Pullout Specimens with High-Yield-Strength Deformed Bars."

LEWIS H. TUTHILL, ROBERT F. ADAMS, SHELLEY N. BAILEY, and RONALD W. SMITH discuss an experience and related testing of "A Case of Abnormally Slow Hardening Concrete."

JEROME M. RAPHAEL explains how "Structural Models Evaluate Behavior of Concrete Dams," describing a method of dead load testing by which the construction stresses as well as the dead load stresses of the completed dam can be determined.

RICHARD W. FURLONG reports on an investigation to determine the "Ultimate Strength of Square Columns Under Biaxially Eccentric Loads," using Whitney's equivalent rectangular stress distribution.

Discussion of papers published in the July, August, and September, 1960, issues will appear in the March JOURNAL.

On the Cover—Shell roof of the Exhibition Palace of the National Center of Industries and Technology, Paris, during the third stage of construction. The project is described in further detail in the News Letter. — Photo courtesy Cement and Concrete Association, London

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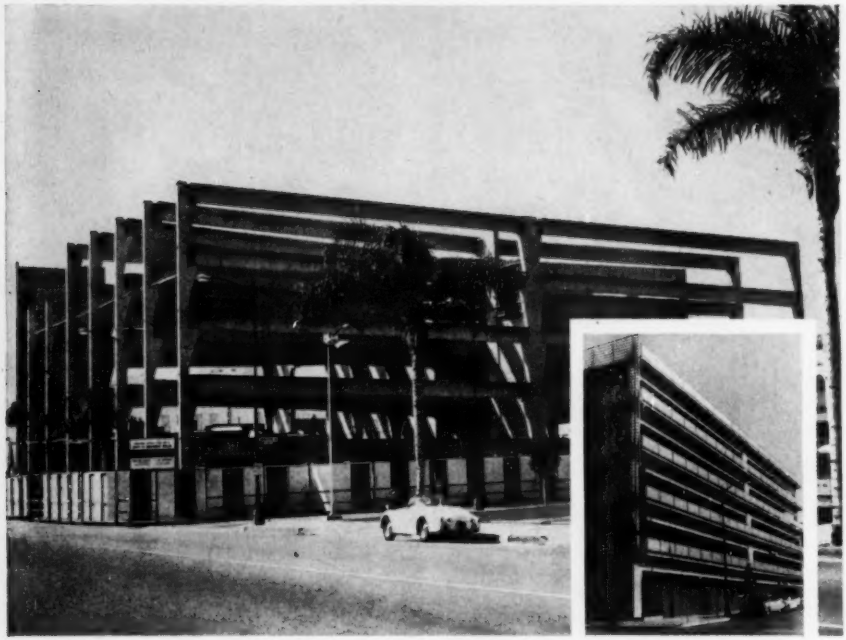

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Title No. 57-42

Precast Grid-Wall for Banque Lambert

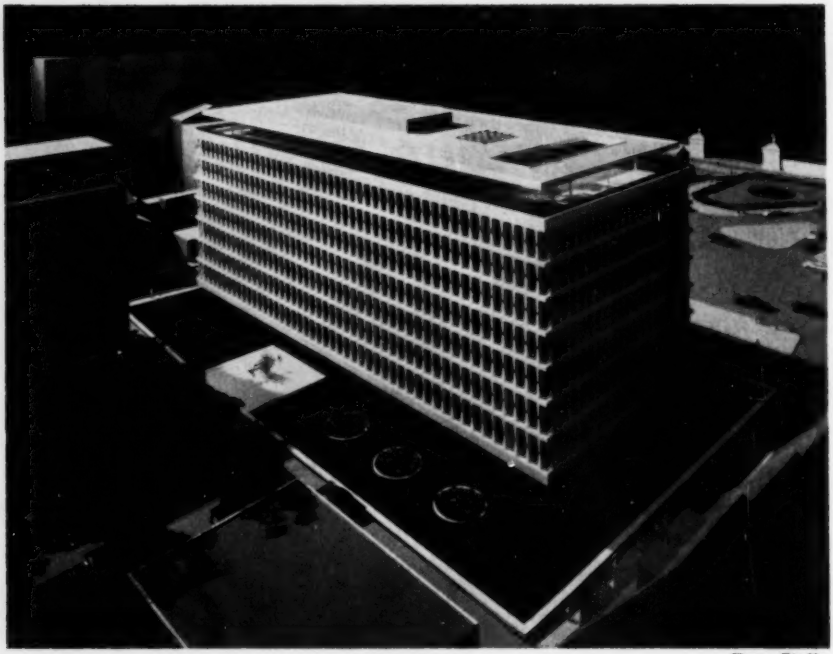
By MATTHYS P. LEVY

A discussion of the design of and construction procedure for a grid-wall consisting of closely spaced precast concrete columns. The merit of placing a hinge at midheight between floors is compared to the difficulties encountered in making a moment connection at the floor level. Examples are given of the present trend in the design of grid-walls. The cost of a grid-wall is compared to the cost of a conventional curtain wall with separate structural columns, and factors affecting cost are evaluated.

■ THE NEW HEADQUARTERS for the Banque Lambert in Brussels presents a new application of precast concrete both as a structural element and as exterior treatment of the building. The building is an eight-story structure facing the Place du Trone in Brussels (Fig. 1). In plan, a typical floor is a rectangle 100 ft x 250 ft with a central core (Fig. 2). Interior columns occur only at each end of the building in line with the core walls. A typical floor consists of a flat ribbed plate 15 in. deep, supported along the periphery of the building by closely spaced columns. Joist construction is used on the two long sides of the building where there is essentially one-way action between the core wall and the outside wall. Two-way dome slabs occur at each end of the building to take advantage of continuity in both directions.

The precast columns on the exterior wall extend from the penthouse floor down to the first floor. At this level they rest on the edge of a thickened slab (Fig. 3). This slab is supported by a peripheral concrete girder set back 9 ft from the outside wall and on the inside by the core wall (or, at the ends of the building, by a concrete beam in line with the core wall).

Steel columns below the first floor support the girder every 39 ft. To assure free rotation of the top of these columns, a hinge is introduced at this point. This design is advantageous because it eliminates torsion in the peripheral girder. The introduction of the hinge also results in a reduction in the size of the column since it is subjected only to an axial load. The columns themselves are encased in concrete, in the form of a pylon. The concrete section of these columns is used to carry that portion of the wind shear not carried by the core walls.



Ezra Stoller

Fig. 1—Model of Banque Lambert building, Brussels

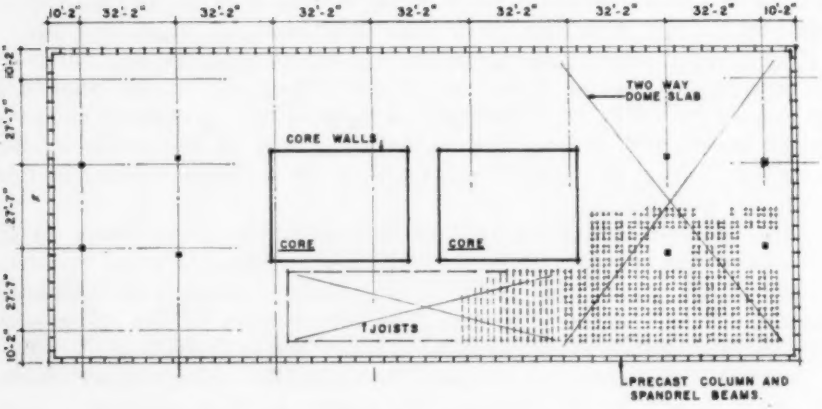


Fig. 2—Typical floor plan

Matthys P. Levy is an associate of Paul Weidlinger, consulting engineer, New York, whom he joined in 1956. He is also visiting critic in architecture, Yale University, and lecturer, Columbia University. Responsible for the design of numerous thin shells and other reinforced concrete structures, he has also worked on research on blast resistant structures. Mr. Levy is author of several articles on shell construction and air inflated structures. He previously worked for Sargent and Lundy; and Severud, Elstad and Krueger.

STRUCTURAL PROBLEM

Although most of the structural details of this building do not require any innovations insofar as design or construction are concerned, one all important detail poses a problem: How can the precast elements of the facade be joined to each other as well as to the floor slab? Or, in other words, how does one provide a moment connection? There are various solutions to this problem. Two examples of joints that have been used in the past are shown on Fig. 4. The first of these illustrates a method used by some German designers, in which tension in the main reinforcement is transferred by bond through a cast-in-place concrete collar. The second shows a connection with a typical base plate welded to the main reinforcement of the column above it and bolted down to the slab. These bolts are welded in turn to the reinforcement projecting from the column below.

However, the basic problem becomes more complicated when the structural concrete column is used as an architectural detail as in the Banque Lambert. In such a case the connection itself must be neat and the exposed concrete must be of a uniform texture and color. This cannot be accomplished by a connection in which precast and in-situ concrete are used together as in the two examples shown on Fig. 4.

Solution

A possible solution of the problem is to place the joint at midheight between floors rather than at the level of the floor slab itself, and

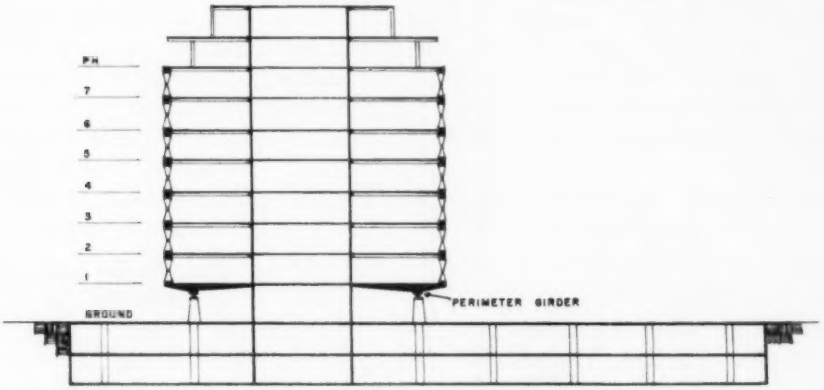


Fig. 3—Typical cross section

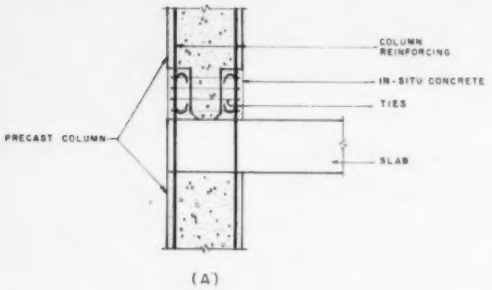


Fig. 4—Typical connections between precast elements

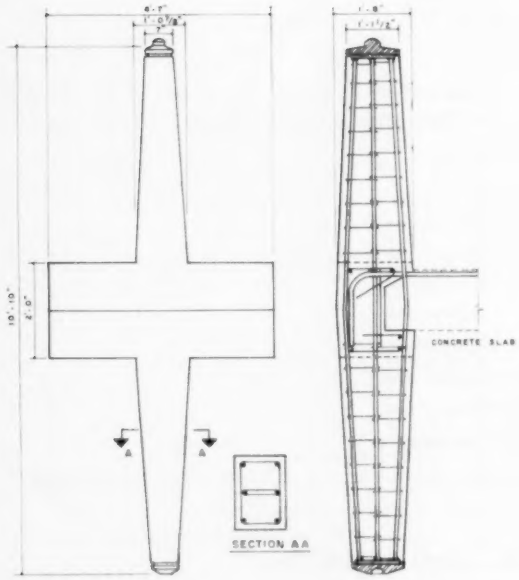


Fig. 5 — Precast concrete column

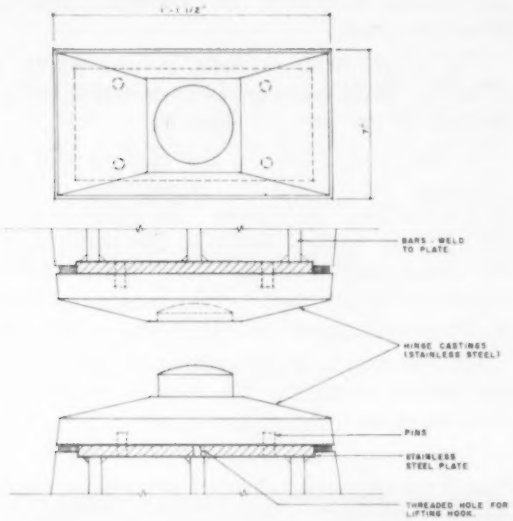


Fig. 6—Hinge details

design the joint to transfer only shears. The difficulties of providing a moment connection are avoided by precasting a column element extending from one joint to the next without interruption at the floor slab. Furthermore, a clean joint is obtained by introducing a stainless steel hinge. The introduction of the hinge has further implications which are discussed later.

The resulting precast element takes the form of a cross 10.8 ft high and 4.6 ft wide (Fig. 5). The horizontal portion is included in the element for two reasons: First, it serves as a spandrel beam for the slab with a notch running the length of the spandrel section acting as a slab seat. Secondly, by precasting the column and spandrel together the whole facade of the building is given a uniform texture and color. A moment connection between the slab and column is achieved by bending dowels from the column into the slab and by running the column reinforcement through the notch. Adjacent spandrel sections are joined to each other through grout keys provided at the ends of each section.

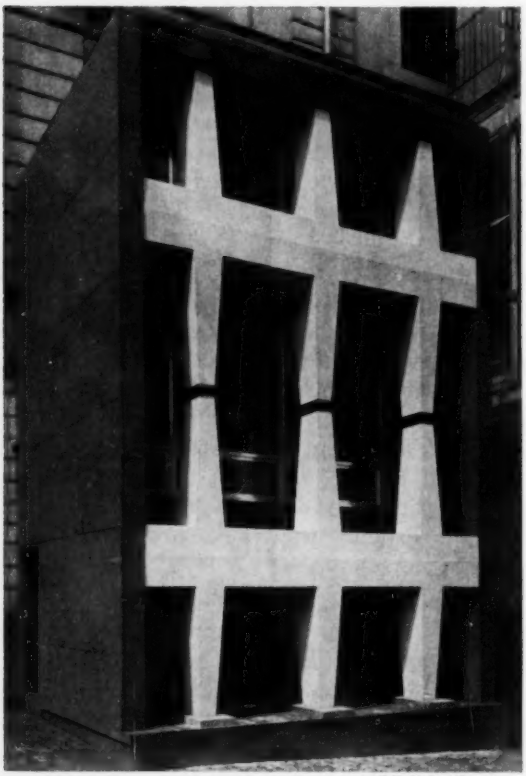
The vertical leg of the precast element is designed as a tied column under a maximum axial load of 152 kips and a maximum shear applied at the hinge of 9 kips. Since the moment in the column varies from zero at the hinge to its maximum value at the floor level, the leg is tapered, to keep the maximum compressive stress approximately constant throughout the height of the column. The principal column reinforcement is welded to stainless steel plates at the top and bottom forming a complete steel cage. A lifting hook is welded to the top plate

for erection purposes. This hook is burned off once the precast element is set in place.

The hinge itself consists of two parts, a male and female casting (Fig. 6). Each casting is connected by pins to a plate at the top and bottom of the precast element.

In the space between the casting and plate shims may be inserted to adjust the length of the element to compensate for shrinkage and manufacturing tolerances. To provide the proper clearance between the top and bottom castings, the rotation of the hinge is calculated on the assumption that the floor above and below are fixed. With this conservative assumption, the maximum rotation is only 0.2 deg, necessitating a minimum lateral clearance of less than 1/16 in.

Schockbeton, a Dutch patented process, is being used to manufacture the precast elements. The process consists of placing a "no-slump" concrete mixture in horizontal steel or wood forms set on a heavy platform. The two end plates of the steel reinforcement cage of the precast elements are held rigidly in a jig attached to the platform. By means of this setup, a ± 3/16 in. tolerance is maintained in the length of the element. The platform is then vibrated at the rate of 250 times a min against a steel anvil. This is a slow vibration compared to a frequency of 3500 impulses per min usually specified in vibration processes. Essentially, the difference between the two lies in the frequency and the intensity of the pulse, the latter being greater for the Schockbeton process. The specified concrete strength for the elements is 6000 psi. Higher strengths are available in Schockbeton but were



S.P.R.L. Bauters

Fig. 7—Full size mock-up of building section

not necessary in the present instance because the dimensions of the elements were prescribed by architectural considerations. The Schockbeton process produces an unusually dense concrete with a basically smooth and hard surface. Other surface treatments are possible depending on the type of aggregate used and on whether etching is used to expose the aggregate on the surface. For this project, white cement and a white river gravel is being used with an etched and polished surface.

Each of the precast elements weighs 3000 lb and present no problems in erection. Once construction has been completed through the first floor, the subsequent sequence on typical floors may be summarized as follows:

1. Erection and temporary bracing of precast elements.
2. Forming and concreting of the slab.
3. Adjustment of the elevation at the top of the pin by placing the required number of shims in the space provided.
4. Erection of next lift, etc.

Full size castings of the column element were made to study the method of precasting, the characteristics of the finish, and the details of fabrication and erection. Six of these elements were assembled into a finished portion of the building complete with floors and glass wall (Fig. 7). As a result of the experience gained in erecting this mock-up, modifications in the hinge detail were found necessary and incorporated into the final design. The mock-up also served to prove the feasibility of the proposed method of erection.

IMPLICATIONS AND DESIGN TREND

In flat slab construction, columns are designed to carry moments introduced by the bending of the slab. Because of the high degree of indeterminacy in a multistory building, the analysis for the column moments becomes a very involved problem. Some simplifying assumptions can be introduced: for example, the usual assumption that the column is fixed at the next higher and lower floors. As shown in Fig. 8a, for a column of constant cross section, this implies a point of inflection at $2H/3$ from the floor in question.

A more elaborate procedure is not normally justified, since the location of the point of inflection does not greatly influence the final moments. Compare, for example, the three cases illustrated in Fig. 8. An extreme assumption of a hinge at the next higher and lower floor yields moments only 7 percent lower than the standard method. The third case with a hinge at a midheight between floors corresponds to the case of the columns described in this paper. Here, the moments are 5 percent above those from the usual assumption. Naturally, greater variations may be obtained depending on the relative stiffness of the floor and column. For all but extreme cases, this variation does not exceed 10 percent which is within the range of engineering approxi-

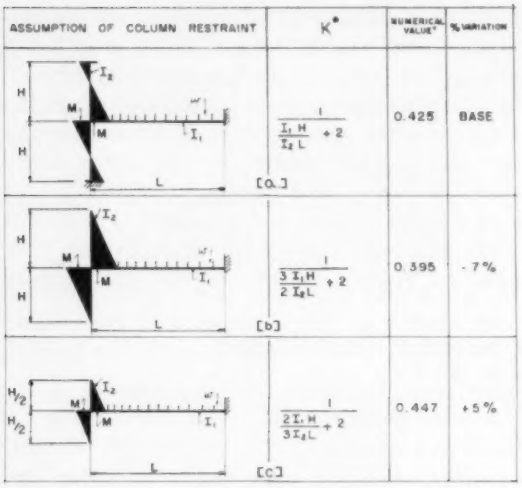


Fig. 8—Computation of column movements

* $M = K\left[\frac{wL^2}{12}\right]$

† BASED ON BANQUE LAMBERT DESIGN

mations. Furthermore, the fact that the columns under consideration are tapered also affects the final moments, but a comparison between an exact analysis which takes into account the tapering and one assuming constant average cross section reveals only a 3 percent difference.

The introduction of a hinge into the column has a double significance. First, it simplifies the analysis because the redundancy of the structure is greatly reduced. In effect, under uniform load applied equally to every floor, it is apparent by geometry that the hinge acts as a simple support, permitting rotation without lateral displacement. Therefore, for analysis, a single floor with the precast element may be isolated from the rest of the structure. This reduces the analysis to one with only two redundants. Second, a column between two hinges forms an element which is easily precast and erected.

There are alternate solutions to the problem of precasting column elements. One method provides a moment connection at the floor. Fig. 9a shows this technique applied to an office building presently under construction. The splice is accomplished by welding the main reinforcing bars to a plate at the top of the column, with space left under the plate for the in-situ slab. A similar plate is provided at the base of the upper column and the two plates are welded together in the field, once the upper column has been leveled and plumbed. A second method proposed for another building eliminates the problem of the moment connection altogether. Here, a cast-in-place slab is sandwiched between two identical column elements in the shape of truncated pyramids. A single post-tensioning rod ties the two elements together and provides the required moment resistance at the floor level. The

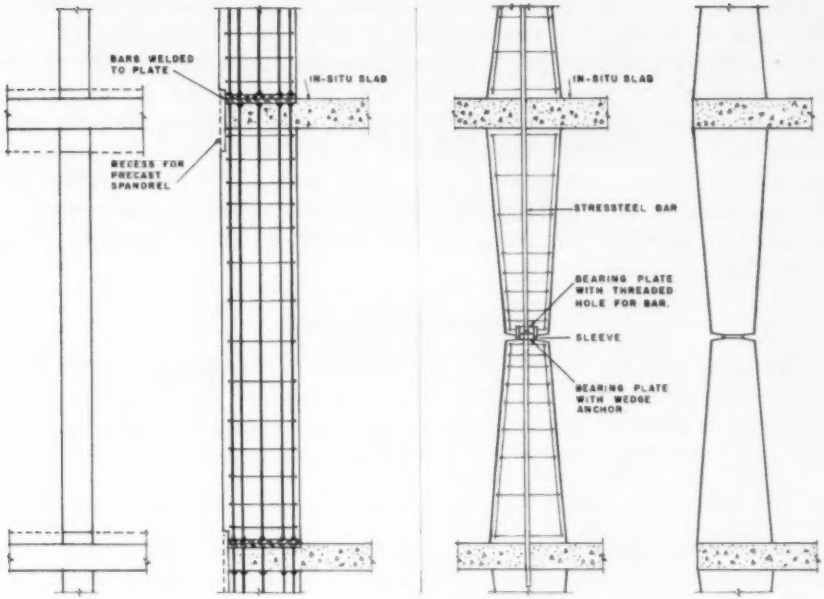


Fig. 9—Alternate precast columns

post-tensioning force is graduated, highest at the top floor where the axial force is the least, lowest at the base of the building where the axial force is the greatest. Here again, there is a hinge at midheight between floors, but the hinge details are simplified by using the end blocks of the post-tensioning rod as bearing plates and a simple sleeve around these blocks to give the necessary shear resistance.

Certainly, there are many other solutions based on the two methods outlined above, but these serve to illustrate the present trend in grid-wall design.

COSTS

Although the design of the grid-wall for this building was governed primarily by esthetic rather than economic considerations, this type of construction compares favorably in cost with typical curtain wall construction: the grid-wall plus the low cost glass wall behind it does not need to cost more than a normal curtain wall plus columns. For the Banque Lambert, the grid-wall is estimated to cost 30 percent more than normal curtain wall construction. This represents less than a 3 percent difference in the total cost of the building. Even this difference would have been completely offset with the use of a smooth natural

finish instead of the etched and polished surface treatment specified. The cost of fabrication and erection of this type of construction may vary from $300 to $600 per cu yd of concrete, depending on the shape and finish of the precast element. Generally, with the proper choice of column spacing, configuration and finish, grid-wall construction may prove more economical than curtain wall construction for concrete buildings. Another consideration is the loss of floor area created by the set back of the glass wall. This is usually offset by the reduction in air conditioning cost from the sun shade effect of the projecting floor slabs.

ACKNOWLEDGMENT

The architects for the project are Skidmore, Owings & Merrill, New York; Enterprises Blaton-Aubert S.A., Brussels, are the contractors and structural engineers. Paul Weidlinger, consulting engineer, New York, served as structural consultant for the project.

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Discussion of this paper should reach ACI headquarters in triplicate by May 1, 1961, for publication in Part 2, September 1961 JOURNAL.

Title No. 57-43

Rectangular Concrete Stress Distribution in Ultimate Strength Design

By ALAN H. MATTOCK, LADISLAV B. KRIZ, and EIVIND HOGNESTAD

An ultimate strength design theory of broad applicability is developed, based on an equivalent rectangular stress distribution in the concrete compression zone and in general accord with the Appendix to the 1956 ACI Building Code. The theory is characterized by simplicity without significant loss of accuracy.

The proposed method of ultimate strength design is applied to a wide variety of structural concrete beams and columns, subject to various combinations of bending and axial load. Calculated ultimate strengths are compared with experimentally determined ultimate strengths for a wide range of variables, and an excellent agreement results.

It is concluded that the proposed extension of the rectangular stress distribution theory permits prediction with sufficient accuracy of the ultimate strength in bending and compression of all types of structural concrete sections likely to be encountered in structural design practice, including odd-shaped sections and other unusual cases.

Part 1 — Review of Basic Assumptions

■ In the October, 1955, report of ACI-ASCE Committee 327 on ultimate strength design as abstracted in the Appendix to the 1956 ACI Building Code (ACI 318-56), ultimate strength design methods were given in specific terms only for the cases most frequently met in design practice. Specific design methods were not given for odd-shaped cross sections and other special cases. Secondly, extensive researches in the United States and abroad have been completed since the Committee 327 report was prepared. A re-evaluation of the design principles involved is therefore desirable. Finally, the rectangular stress distribution theory has become widely used in design practice and in general accord with the 1956 Code. The extent to which this simple theory can be safely extended to unusual design cases has previously not been thoroughly studied.

At the request of Subcommittee 15 on ultimate strength design of ACI Committee 318, Standard Building Code, and in the light of recent research findings, the authors have studied the rectangular stress distribution theory as commonly used in design practice under the 1956 Code. In addition, they have extended this theory into a *generally applicable* and reasonably simple design tool based on a single set of assumptions.

This paper presents such assumptions and substantiating evidence for consideration by ACI Committee 318 and the ACI membership as a contribution toward development of a future revision of the ACI Building Code. In this manner, the recommendations and views expressed are those of the authors and do not necessarily reflect the collective judgment of Committee 318.

ASSUMPTIONS IN ULTIMATE STRENGTH DESIGN

By use of the following general assumptions, ultimate strength of sections subjected to combined bending and axial load can be predicted with adequate accuracy, even for odd-shaped cross sections and other unusual cases:

1. At ultimate strength, a concrete stress of intensity 0.85 times the concrete cylinder strength may be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at a distance k_1c from the region of maximum compressive strain. The distance c from the region of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The fraction k_1 is taken as 0.85 for concrete cylinder strengths up to 4000 psi and is reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi.
2. Tensile strength of the concrete may be neglected in flexural calculations.
3. Strain in the concrete at the various section levels may be assumed directly proportional to the distance from the neutral axis. Except in anchorage regions, strain in reinforcing bars may be assumed equal to the tensile or compressive strain in the concrete at the same distance from the neutral axis as the centroid of each bar or group of bars considered.
4. The maximum strain at an extreme edge of the concrete compression zone may be assumed equal to 0.003.
5. Stress in reinforcing bars below the yield point stress for the grade of steel used may be taken as 30,000,000 psi times the steel strain. For strain greater than that corresponding to the yield point stress, the reinforcement stress may be considered independent of strain and equal to the yield point stress.

USE OF SIMPLIFIED EQUATIONS

The equations for ultimate strength design given in the Appendix to the 1956 ACI Building Code, ACI 318-56, can almost all be derived from the assumptions set out above. Certain of the Code equations for the design of columns contain additional simplifying assumptions.

Alan H. Mattock (ACI member) joined the staff of the Portland Cement Association Research and Development Laboratories, Skokie, Ill., in 1957. After graduation from London University, he served with the Fire Research Station at Elstree, England, and in the Lancashire County Bridgmaster's design office. He returned to study and was awarded MS (Eng), London University, in 1949. For the next 3 years he was a district engineer in British Guiana, South America, where the principal work was the construction of sea and river walls, and drainage and irrigation works.

From 1952 to 1957 Dr. Mattock was a lecturer at Imperial College, University of London, having charge of the Concrete Technology Laboratory; while there he carried out research and consulting work, and in 1955 was awarded PhD, London University. Dr. Mattock is a member of ACI-ASCE Committee 323, Prestressed Concrete, and also of ACI-ASCE Committee 328, Limit Design.

Ladislav B. Kriz (ACI member) graduated in civil engineering from the University of Illinois in 1957, with previous schooling at the Technical University in Prague, Czechoslovakia, and at the School of Civil Engineering in Madrid, Spain. His undergraduate studies were interrupted by brief periods of employment as draftsman with Zublin-Perriere, Paris, France; as a designer with Byrnes Associates, New York, N. Y.; and a 4-year tour of duty as surveyor in the U. S. Air Force.

He joined the staff of the Structural Section of the Portland Cement Association Research and Development Laboratories in 1957. During his employment with the association, he earned his MS degree from Northwestern University.

Eivind Hognestad (ACI member), manager, Structural Development Section, Portland Cement Association Research and Development Laboratories, is well known to ACI JOURNAL readers. Prior to joining the PCA staff in 1953, Dr. Hognestad was research associate professor of theoretical and applied mechanics at the University of Illinois.

He obtained his MS degree in civil engineering from the University of Illinois and the degrees of civil engineer and doctor of technical sciences from the Norwegian Institute of Technology.

Dr. Hognestad is chairman of ACI-ASCE Committee 326, Shear and Diagonal Tension; a member of ACI Committee 318, Standard Building Code, and ACI Committee 336, Combined Footings.

For the straight-forward cases, for which these Code equations have been derived, the equations are entirely adequate and may be used with confidence. It should be noted however, in using Eq. (A3) from the Appendix to the 1956 ACI Code for the design of a beam with compression reinforcement, that the stress in the compression steel at ultimate strength should be checked to confirm that yielding of the steel has in fact occurred.

In unusual problems of design, the basic assumptions given may be used directly in everyday practice. Cases not susceptible to direct mathematical solution may be treated by suitable iteration procedures. In mathematically complex cases, calculations based on the basic assumptions may be used together with experimental data to develop and verify simplified procedures for design office use, such as those proposed in the writings of P. M. Ferguson and C. S. Whitney.

Notation

Wherever possible the notation used in the ultimate strength design appendix to the ACI Building Code (ACI 318-56) has been used in this paper. For convenience, the notation is summarized as follows:

A_c = net concrete cross section, or area of segment of circle covered by equivalent concrete stress distribution

A_s = area of tensile reinforcement

A_s' = area of compressive reinforcement

A_{sf} = steel area required to develop compressive strength of overhanging flange in T-sections

A_{st} = total area of longitudinal reinforcement

a = depth of equivalent rectangular stress distribution

b = width of a rectangular section, or over-all width of flange in T-sections

b' = width of web in T-sections

C = resultant concrete compressive force

c = distance from extreme compressive fiber to neutral axis at ultimate strength

D = diameter of a circular section

d = distance from extreme compressive fiber to centroid of tensile reinforcement

d' = distance from extreme compressive fiber to centroid of compressive reinforcement

E_s = modulus of elasticity of reinforcing steel

e = eccentricity of axial load, measured from the centroid of tensile reinforcement, unless otherwise specified

F = ratio of change in strain of steel of prestressed beam to change in strain of concrete at the level of the steel

F_{sc} = resultant steel compressive force

F_{st} = resultant steel tensile force

f_c' = 6 x 12-in. concrete cylinder strength

f_s = stress in tensile reinforcement at ultimate strength

f_s' = stress in compressive reinforcement at ultimate strength

f_y = yield point stress of tensile reinforcement

f_y' = yield point stress of compressive reinforcement

G_c = center of action of concrete compressive force

G_{sc} = center of action of steel compressive force

G_{st} = center of action of steel tensile force

k_u = c/d

k_1 = ratio of average stress to maximum stress

k_2 = ratio of depth to resultant of concrete compressive force, to depth of neutral axis

k_3 = ratio of maximum stress to 6 x 12-in cylinder strength, f_c'

M_{ult} = ultimate moment of resistance

m = plastic modular ratio, $\varepsilon_u E_s / 0.85 k_1 f_c'$

P_u = ultimate strength of eccentrically loaded member

P_o = ultimate strength of concentrically loaded member

p = tensile steel ratio, A_s/bd

p' = compressive steel ratio, A_s'/bd

p_b = steel ratio at balanced ultimate strength condition in a beam without compression reinforcement

p_{bb} = steel ratio at balanced ultimate strength condition in beam with both tension and compression reinforcement

p_f = $A_{sf}/b'd$

p_w = tension steel ratio for T-beams, $A_s/b'd$

p_{wb} = tension steel ratio for T-beam with balanced ultimate strength condition

q = tension reinforcement index, pf_y/f_c'

q' = compression reinforcement index, $p'f_y'/f_c'$

q_b = tension reinforcement index for balanced ultimate strength conditions, $p_b f_y/f_c'$

q_w = tension reinforcement index for T-beams, $p_w f_y/f_c'$

q_{wb} = tension reinforcement index for T-beams with balanced ultimate strength condition, $p_{wb} f_y/f_c'$

t = flange thickness in T-beams, also total depth of rectangular section column

α = inclination of neutral axis to horizontal in nonsymmetrical section
ε = strain
ε_e = concrete tensile strain at ultimate at level of steel in prestressed beam
ε_{cp} = concrete precompression strain at level of steel in a prestressed beam
ε_u = maximum concrete compression strain at ultimate beam strength
ε_s = tensile steel strain at ultimate beam strength
ε_y = tensile steel yield strain
ε_s' = compression steel strain at ultimate strength
ε_y' = compression steel yield strain
ε_{se} = effective steel prestrain
ε_{sn} = strain in steel distance, a_n, from neutral axis
θ = half included angle between two upper faces of beams with triangular compression zones

CONCRETE STRESS DISTRIBUTION

The general form of the concrete compression stress distribution at ultimate strength in a reinforced concrete member is shown in Fig. 1. The properties of the "stress block" are represented by the following coefficients:

k_1 = ratio of average stress to maximum stress
k_2 = ratio of depth to resultant of compressive force, to depth to neutral axis
k_3 = ratio of maximum stress to 6 x 12-in. cylinder strength, f_c'

Historical background

The use of a design theory based on the ultimate strength of sections is in effect a return to the original concept of design, in that early design formulas were empirical, being based on the failure loads of typical elements as found by experiment.

The first published ultimate load theory was that of Koenen[1] who in 1886 assumed a straight line distribution of concrete stress and a neutral axis at middepth. Since that time about 30 theories have been published. The salient points of many of these theories were set out in *Bulletin* No. 399,[8] University of Illinois Engineering Experiment Station.

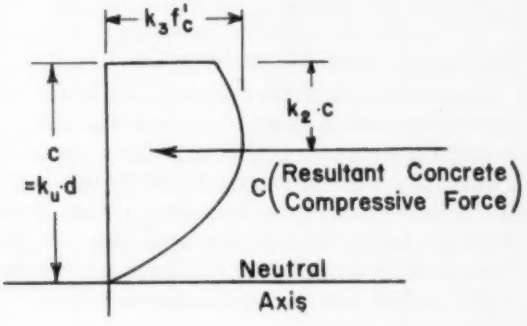

Fig. 1—Concrete stress distribution at ultimate strength

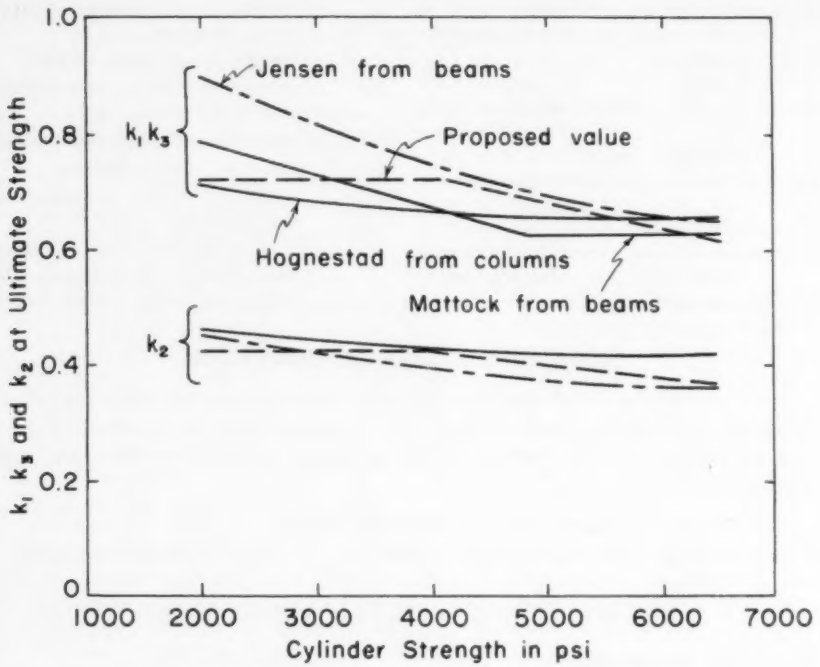

Fig. 2—Design values from tests of reinforced concrete

Many different distributions of stress in the concrete compression zone have been suggested, but in the light of recent experimental[2] and analytical[3] investigations, it is considered that the use of the "equivalent rectangular stress block" in design calculations will yield sufficiently accurate results and will at the same time lead to considerable simplification of design calculations. The use of a rectangular concrete compression stress block was first proposed by von Emperger[4] in 1904, and since that time by several other engineers, the best known in this country being C. S. Whitney, whose paper[5] in 1937 was a notable contribution to the literature of ultimate strength design.

Reinforced concrete investigations

Until recently, most of the available information regarding stress distribution in concrete was derived from tests of reinforced concrete members. In the early 1930's the extensive ACI investigation[6] of concentrically loaded columns led to the addition law which states that the ultimate strength of a column is equal to 85 percent of the cylinder strength times the concrete area plus the yield stress of the longitudinal steel times its area. Thus, for concentrically loaded columns, the value of $k_3 = 0.85$ was derived experimentally.

Several papers on stress distribution and ultimate strength design were published in Europe during the 1930's, and these were followed by the studies of ultimate strength in this country by Whitney.[5] His analytical approach involved an equivalent rectangular stress block with a maximum stress of 0.85 f_c', thus his value of k_3 was 0.85.

In 1943 an extensive study of the ultimate strength of beams was presented by Jensen.[7] He proposed a trapezoidal idealized stress distribution, and derived the properties of this trapezoid as a function of cylinder strength by analysis of the observed ultimate strength of reinforced concrete beams. On this basis he found the values of k_1k_3 and k_2 shown in Fig. 2.

A study of the ultimate strength of eccentrically loaded columns was reported by Hognestad[8] in 1951. The stress distribution used consists of a rising parabola and a descending straight line after the maximum stress. From the results of concentrically loaded columns, a maximum stress equal to 0.85 f_c' was chosen, that is, $k_3 = 0.85$. The slope of the descending straight line was chosen so as to give the best statistical agreement between calculated and observed column strengths in 120 tests. The corresponding values of k_1k_3 and k_2 are also shown in Fig. 2.

Additional experimental evidence as to the parameters of the concrete stress block was presented in 1956.[9] Using the measured values of depth to the neutral axis at failure and of the ultimate moment of resistance, and assuming a safe limiting value 0.43 for k_2, values of k_1k_3 were calculated for 69 beams tested at Imperial College, University of London, and at the University of Illinois. Based on these results the values of k_1k_3 labeled "Mattock" in Fig. 2 were proposed as suitable for ultimate strength design.

Plain concrete investigations

In recent years various tests have been carried out on plain concrete specimens, using special testing techniques, in an effort to obtain a true picture of the stress-strain relationship for concrete during loading to failure. Independently, but almost simultaneously, tests of eccentrically loaded prismatic concrete specimens were carried out in the Portland Cement Association laboratories[10] and by Rüsch[11] at the Munich Institute of Technology.

The specimen used in the PCA tests is shown in Fig. 3. The two thrusts P_1 and P_2 were varied independently, in such a manner that the neutral axis was maintained at the bottom face of the specimen throughout the test. By equating the internal and external forces and moments, it was possible to calculate the values of k_1k_3 and k_2 directly.

In the Munich tests, groups of about five identical prisms were tested with a different and constant eccentricity for each test. By plotting, for all specimens within one group, strain measured at an outside

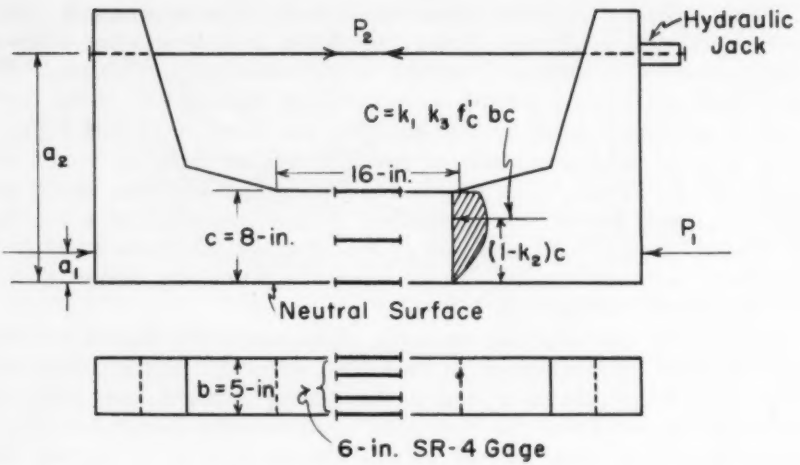

Fig. 3—PCA eccentric load specimen

face at ultimate strength versus applied eccentricity, the eccentricity corresponding to a position of the neutral axis at an edge of the section was determined. By applying statistical methods, the magnitude and position of the internal concrete force at ultimate strength was determined.

The parameters of the concrete stress block obtained at the two laboratories are summarized and compared in Fig. 4 with the values proposed in this paper. The PCA tests with sand-gravel aggregates have been reported in detail elsewhere,[10] a detailed report on the tests by J. A. Hanson with lightweight aggregates has not yet been published. The German tests[11] were made with sand-gravel aggregates. It can be seen that test results for sand-gravel concretes are in good agreement, and that the radical change in aggregate type to lightweight materials caused only a minor change in the stress distribution properties.

Analytical investigations

In a recent analytical paper[3] the problem of ultimate flexural strength of reinforced concrete members was reduced to finding the maximum value of a load function expressed in terms of the internal resisting forces of the loaded member. Assuming only that concrete stress, f, is some function of strain, ε, given by $f = F(\varepsilon)$, and that plane sections remain plane during bending, the moment of resistance of a reinforced concrete section was expressed in terms of the extreme edge concrete strain, ε_c, the reinforcement yield point, f_y, and the dimensional properties of the cross section. This expression for moment was differentiated with respect to the extreme edge concrete strain and equated to zero. The resulting equation demonstrates that, when the maximum moment in a rectangular reinforced concrete beam failing in tension under

symmetrical bending is reached, then the concrete stress distribution is such that the total compressive force is equal to that obtained from an equivalent rectangular stress block in which the stress is equal to the actual stress in the extreme compression fibers, and which has a depth equal to twice the distance from the extreme compressive fibers to the center of action of the resultant concrete compressive force. From a study of concrete stress-strain curves obtained in the PCA eccentrically loaded prism investigation,[10] it was deduced that the relationship between concrete stress at extreme compressive fiber, f_u, and the cylinder strength, f_c', can be expressed closely as

$$f_u = 5 f_c'^{0.8}$$

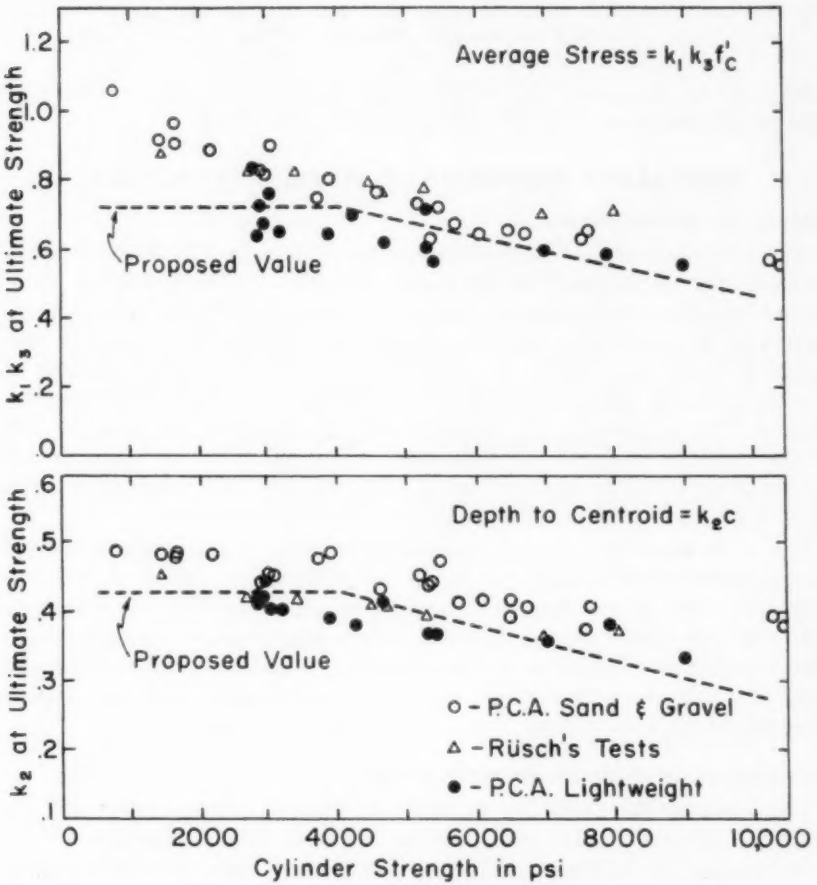

Fig. 4—Properties of concrete stress distribution at ultimate strength determined from tests on plain concrete

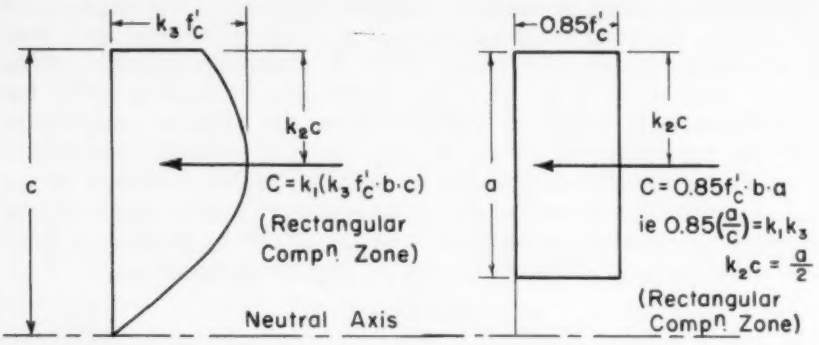

Fig. 5a (left)—Actual concrete stress distribution. Fig. 5b (right)—Equivalent rectangular stress distribution

However, for purposes of practical application a conservative straight-line relationship, $f_u = 0.85 f_c'$ was proposed.

EQUIVALENT RECTANGULAR STRESS DISTRIBUTION

Nature of approximation

The essentials of the approximation are shown in Fig. 5 and may be summarized as follows: At ultimate strength, the compressive stress in the concrete compression zone is, for purposes of computation, assumed to be uniformly distributed from the region of maximum compressive strain to a depth, a, measured in a direction perpendicular to the neutral axis, where the depth, a, is less than the depth to the neutral axis, c. The uniformly distributed stress is taken to be equal to 0.85 of the cylinder strength, f_c'. The ratio, a/c is taken to be equal to 0.85 for concrete cylinder strength up to 4000 psi, and thereafter is reduced by 0.05 for each 1000 psi of strength in excess of 4000 psi.

For a rectangular concrete compression zone the average concrete compressive stress $k_1 k_3 f_c'$ is equal to $0.85(a/c) f_c'$, that is $k_1 k_3 = 0.85(a/c)$, also $k_2 = \frac{1}{2}(a/c)$. Since k_3 is taken as 0.85 f_c', it follows that $a/c = k_1$ and $a = k_1 c$. For the values of a/c proposed above, $k_1 k_3$ and k_2 will have values as indicated in Fig. 2 and 4. It is seen that the proposed values for $k_1 k_3$, and k_2 correspond closely with the values determined by experiment.

Assumptions in ultimate strength design

The assumptions necessary in ultimate strength design were set out earlier in this paper, and will now be discussed item by item.

The nature of the first assumption has already been discussed above.

The second assumption is very nearly correct; any tension zone which does exist in the concrete at ultimate strength, in a normally propor-

tioned reinforced concrete section, is very small indeed; and its lever arm is also small. It is therefore reasonable to neglect any contribution by concrete tension to the ultimate moment of resistance of a section.

The third assumption is not strictly correct for a reinforced concrete section after cracking, since the strain in the concrete on the tension side of the neutral axis will vary considerably, at any given level, due to cracking. If, however, we measure the extension per unit length of

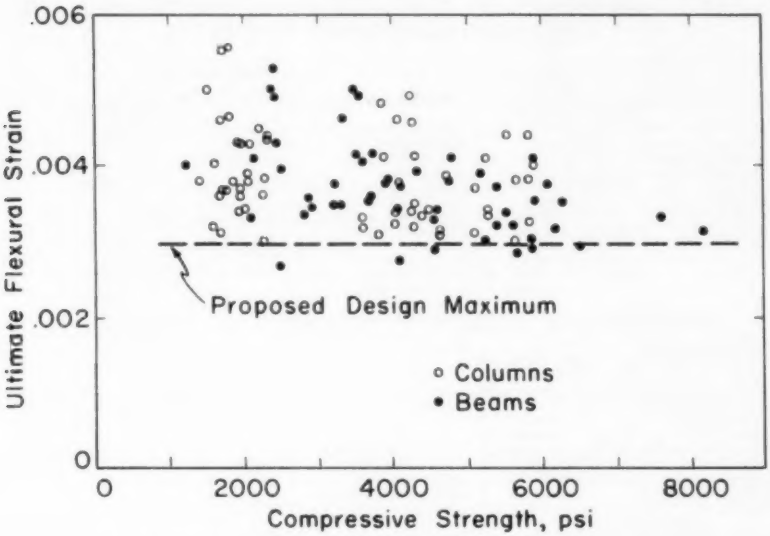

Fig. 6a—Ultimate strain from tests of reinforced members

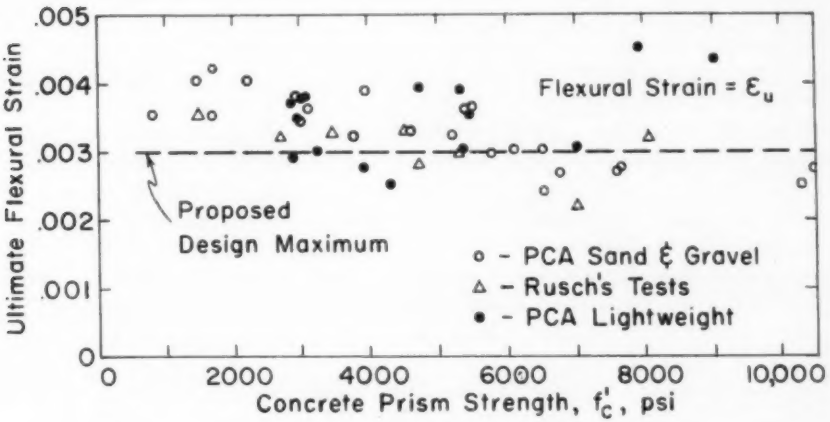

Fig. 6b—Ultimate strain from tests of plain concrete specimens

a gage length including several cracks, we find that this "apparent" tensile strain varies linearly with distance from the neutral axis. If for a particular section, we measure the compression strains and apparent tensile strains using the same gage length, then the distribution of these strains will be very close to linear. By experiment it has also been found that for the normal type of deformed bar reinforcement, the strain in the steel is nearly equal to the strain in the adjacent con-

TABLE 1—SUMMARY OF COMPARISONS OF CALCULATED AND EXPERIMENTAL ULTIMATE STRENGTHS

Type of member	Loading type and number tested	Failure mode	Average value: Test ultimate strength / Calc. ultimate strength	Standard deviation
Reinforced concrete rectangular beams reinforced in tension	Simple bending, 59 beams	Controlled by crushing of concrete	All beams: 1.06 Beams with $f_c' \geq$ 2000 psi: 1.02	0.113 0.083
Reinforced concrete rectangular beams reinforced in tension and compression	Simple bending, 33 beams	Controlled by yield of tension reinforcement	Beams using structural or intermediate grade steel: 1.04 Beams using high strength alloy steel: 1.21	0.042 0.170
	Simple bending, 11 beams	Controlled by crushing of concrete and yield of compression steel	All beams: 1.07	0.072
Prestressed beams with well bonded tendons	Simple bending, 32 beams	Varied	All beams: 1.03	0.077
Prestressed beams with unbonded tendons	Simple bending, 24 beams	Varied	All beams: 1.05	0.094
Symmetrical reinforced concrete T-beams reinforced in tension	Simple bending, 15 beams	Controlled by yield of the reinforcement	All beams: 1.11	0.100
Unsymmetrical reinforced concrete T-beams reinforced in tension	Simple bending, 12 beams	Varied	All beams: 1.10 Beams controlled by yield of steel: 1.04	0.138 0.047
Reinforced concrete beams with a triangular compression zone reinforced in tension	Simple bending, 6 beams	Controlled by crushing of concrete	All beams: 1.10	0.069
Reinforced concrete rectangular columns	Axial loading, 16 columns	Controlled by crushing of concrete and yield of steel	All columns: 1.00	0.074
Reinforced concrete rectangular columns	Axial loading plus bending about one principal axis, 86 columns	Varied	All columns: 0.97	0.059
Reinforced concrete rectangular columns	Axial loading plus bending about two principal axes, 10 columns	Controlled by crushing of concrete and yield of some of the steel	All columns: 0.99	0.046
Reinforced concrete circular section columns	Eccentric load, 30 columns	Controlled by crushing of concrete and yield of some of the steel	All columns: 1.05	0.060

crete. The second assumption is therefore sufficiently accurate for the purpose of design calculations.

The maximum concrete compressive strain at ultimate strength has been measured in many tests of plain and reinforced concrete members. In Fig. 6a test results are plotted which were obtained at the University of Illinois[8,12] in tests of reinforced concrete columns and beams, while in Fig. 6b are shown values of maximum concrete compressive strain measured in the PCA[2] and Munich[11] tests of plain concrete. It can be seen that the value of 0.003 in. per in. proposed for design is a reasonably conservative value.

The implication of the fifth assumption is that the effect of strain hardening of the steel above the yield point is neglected. Compatibility of strains can no longer be relied on at high steel strains beyond the yield point, also the stress-strain curve beyond the yield point is not (and probably cannot conveniently become) stipulated in American specifications for manufacture of reinforcement. It is therefore considered unwise in design to rely on obtaining an increase in ultimate strength through strain hardening of the reinforcement.

SUMMARY OF COMPARISONS WITH EXPERIMENTS

Development of design equations and comparison with test data are presented in detail in Part 2 and the appendix, respectively. A summary is presented below.

The correctness of the equation for ultimate strength of a reinforced concrete beam reinforced only in tension, with the strength controlled by tension, is verified by R. C. Elstner's statistical analysis of 364 tests of this type of beam. This analysis leads to a value of 0.593 for the coefficient of q in the equation, as against a value of 0.59 derived from the assumptions set out in this paper, the differences being insignificant.

The comparisons of calculated and experimental ultimate strengths for other types of member and conditions of loading are summarized in Table 1. The data contained in the table were obtained from tests of 334 structural concrete elements of nine different types, in which the elements were subjected to four different combinations of flexure and axial loading. The tests included cases in which the concrete compression zone at ultimate strength was segmental, trapezoidal, or triangular in shape, in addition to the more common rectangular shape. It will be seen that there is close agreement between the experimental and calculated ultimate strengths. The mean value of "Test Ultimate Strength"/"Calculated Ultimate Strength" for all 334 cases considered is 1.037, and the standard deviation is 0.097.

CONCLUDING REMARKS

The validity of the basic assumptions, particularly the use of an equivalent rectangular stress distribution as proposed herein, and their

applicability to the calculation of the ultimate strength of structural concrete sections has been confirmed by two findings:

1. The parameters proposed for use in ultimate strength design, as calculated from the basic assumptions, are in close agreement with the values of these parameters determined from tests of plain and reinforced concrete.

2. There is excellent agreement between the ultimate strengths of a wide range of structural concrete members determined experimentally, and the calculated ultimate strengths of the members based on the assumptions set out in this paper.

It is concluded, therefore, that the proposed method of ultimate strength design permits prediction with sufficient accuracy of the ultimate strength in bending, in compression, and in combinations of the two, of all types of structural concrete sections likely to be encountered in practice.

Part 2 — Design Equations

DESIGN OF RECTANGULAR BEAMS IN BENDING

Beams reinforced in tension only

Ultimate strength controlled by yielding of reinforcement—In this case, steel stress at ultimate strength, f_s, equals the yield stress f_y.

From equilibrium of internal forces in Fig. 7

$$0.85k_1f_c'\,b\,c = A_s f_y \qquad (1.1)$$

From equilibrium of internal and external moments

$$M_{ult} = A_s f_y(d - k_2c) \qquad (1.2)$$

Solving Eq. (1.1) and (1.2) and substituting $q = p f_y/f_c'$:

$$M_{ult} = A_s f_y d \left(1 - \frac{k_2}{0.85k_1} q\right) \qquad (1.3)$$

or

$$M_{ult} = b\,d^2 f_c' q \left(1 - \frac{k_2}{0.85k_1} q\right) \qquad (1.3A)$$

Since $k_1 = 2k_2$, we may write

$$M_{ult} = A_s f_y d(1 - 0.59q) \qquad (1.4)$$

or

$$M_{ult} = b\,d^2 f_c' q(1 - 0.59q) \qquad (1.4A)$$

Eq. (1.4A) is, of course, identical with Eq. (A1) in the Appendix to ACI 318-56.

The correctness of Eq. (1.4A) was demonstrated statistically by R. C. Elstner. Using the general concrete stress distribution parameters

k_1,k_2,k_3, Eq. (1.3A) may also be written:

$$M_{ult} = bd^2 f_c' q \left(1 - \frac{k_2}{k_1 k_3} q\right) \quad (1.5)$$

Examining the results of 364 beam tests by the method of least squares k_2/k_1k_3 is given by:

$$\frac{k_2}{k_1 k_3} = \frac{-\Sigma \frac{M_{ult}}{bd^2 f_c'} q^2 + \Sigma q^3}{\Sigma q^4} \quad (1.6)$$

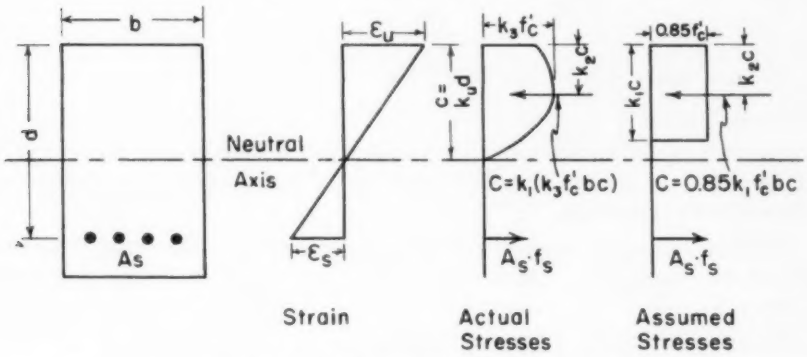

Fig. 7—Conditions at ultimate strength

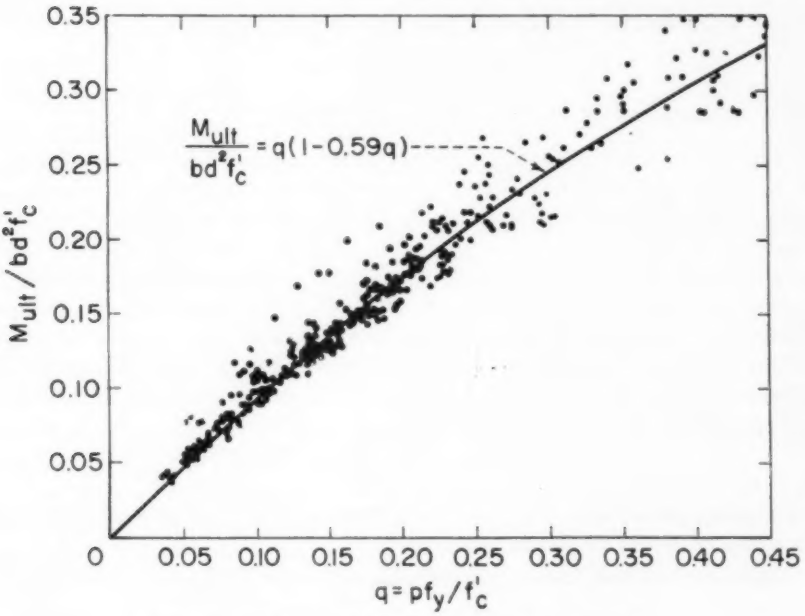

Fig. 8—Tests of 364 beams controlled by tension

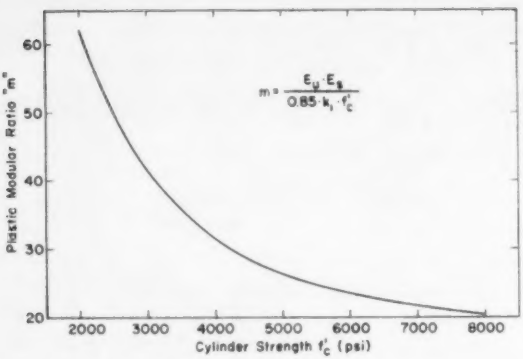

Fig. 9—Variation of plastic modular ratio *m* with concrete cylinder strength

Using this equation Elstner found $k_2/k_1k_3 = 0.593$. In Fig. 8, the curve of Eq. (1.4) is drawn, together with the 364 test results considered in Elstner's investigation.

Ultimate strength controlled by compression—In this case crushing of the concrete takes place while the steel stress is below the yield point.

From equilibrium of internal forces:

$$0.85\, k_1\, f_c'\, b\, c = A_s\, f_s \qquad (1.1A)$$

From equilibrium of internal and external moments:

$$M_{uII} = A_s\, f_s (d - k_2 c) \qquad (1.2A)$$

From assumption of linear strain distribution:

$$k_u = \frac{c}{d} = \frac{\varepsilon_u}{\varepsilon_s + \varepsilon_u} \qquad (1.7)$$

Solving Eq. (1.1A) and (1.7) for k_u:

$$k_u = \sqrt{pm + \left(\frac{pm}{2}\right)^2} - \frac{pm}{2} \qquad (1.8)$$

where $m = (E_s\, \varepsilon_u) / (0.85\, k_1\, f_c')$ as shown in Fig. 9.

The ultimate moment of resistance of a section may be calculated by substituting the value of k_u obtained from Eq. (1.8) in the following equation:

$$M_{uII} = (0.85\, k_1\, f_c')\, b\, d^2\, k_u (1 - k_2\, k_u) \qquad (1.9)$$

Balanced conditions—In this case simultaneous yielding of steel and crushing of concrete takes place.

The balanced steel ratio, p_b, is obtained by solving Eq. (1.1) and (1.7):

$$p_b = 0.85 k_1 \frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u} \frac{f_c'}{f_y} \qquad (1.10)$$

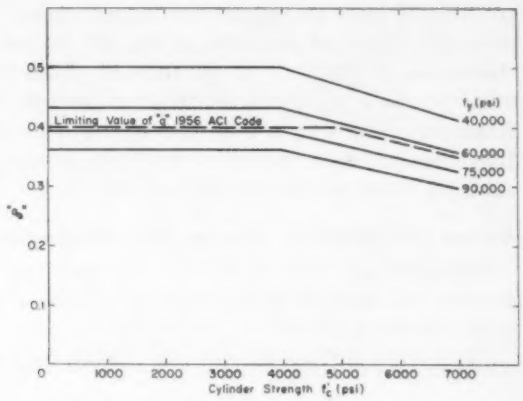

Fig. 10—Steel ratio q_b for balanced ultimate strength

We may also write:

$$q_b = p_b \frac{f_y}{f_c'} = 0.85\, k_1 \frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u} \qquad (1.11)$$

or, substituting for ε_u and ε_y

$$q_b = 0.85\, k_1 \left(\frac{90{,}000}{f_y + 90{,}000}\right) \qquad (1.11A)$$

The ultimate moment for balanced failure may then be written:

$$M_{ult} = (0.85\, k_1 f_c')\, b\, d^2 \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u}\right)\left[1 - k_2 \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u}\right)\right] \qquad (1.12)$$

Alternatively the value of q_b found using Eq. (1.11A) may be substituted in Eq. (1.4).

Fig. 10 shows the balanced steel ratios q_b for various strengths of concrete and steel. Also plotted in Fig. 10 is the limiting value of q to be used in design of sections prescribed by ACI 318-56. It is seen that the limiting value prescribed is safe for steel yield points not exceeding 60,000 psi as assumed in the Code, but the limit is in fact greater than q_b for high steel stresses. If it is considered desirable for design purposes to establish a limiting value of q less than q_b even for high steel stresses, then it is proposed that this limiting value be expressed as a fraction of q_b and not in the form in current use. If, however, a simple direct expression for the limiting value of q is desired, then the following is proposed:

"For $f_c' < 4000$ psi, $q_{lim} = 80/\sqrt{f_y}$; for concrete strengths greater than 4000 psi reduce q_{lim} by 0.02 for each 1000 psi." This simplified expression ensures that q_{lim} will be between 70 percent and 80 percent of q_b for an extremely wide range of concrete and steel strengths.

In Table A-1 of the appendix a comparison is made between the ultimate moment calculated using Eq. (1.9) and the ultimate moment meas-

ured in 59 tests on singly reinforced beams failing in compression. The average value of M_{test}/M_{calc} for all beams is 1.06 and the standard deviation is 0.113. It is of interest from a practical design point of view to note that the average value of M_{test}/M_{calc} for beams made from concrete with a cylinder strength of 2000 psi or over is 1.02, and that the standard deviation for this group of beams is 0.083. A histogram of these results is plotted in Fig. 11.

Beams reinforced in tension and compression

Ultimate strength controlled by yielding of tension reinforcement— In this case tensile steel stress at ultimate strength, f_s, is equal to the yield stress, f_y.

From equilibrium of internal forces in Fig. 12:

$$0.85\, k_1 f_c' \, b\, c + A_s' f_s' = A_s f_y \qquad (2.1)$$

From equilibrium of internal and external moments:

$$M_{ult} = 0.85\, k_1 f_c' \, b\, c(d - k_2 c) + A_s' f_s' (d - d') \qquad (2.2)$$

In most normal doubly reinforced beams the compression steel is close to the face of maximum concrete compression. The strain in the steel

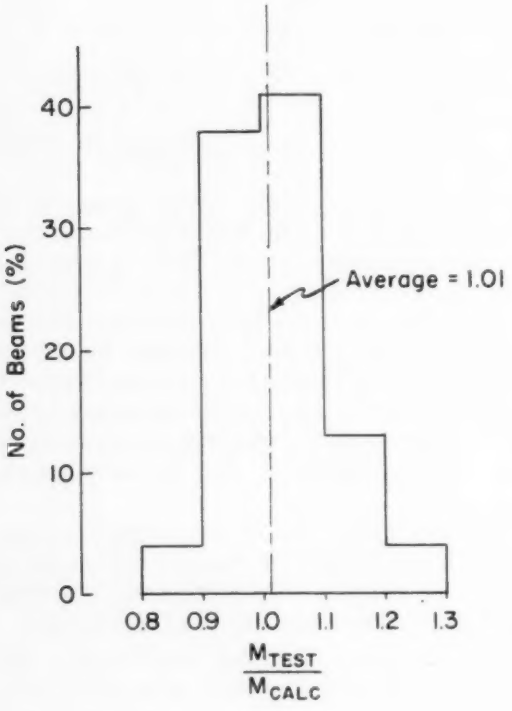

Fig. 11 — Histogram of (M_{test}/M_{calc}) for compression failures of singly reinforced concrete beams with concrete strength greater than 2000 psi

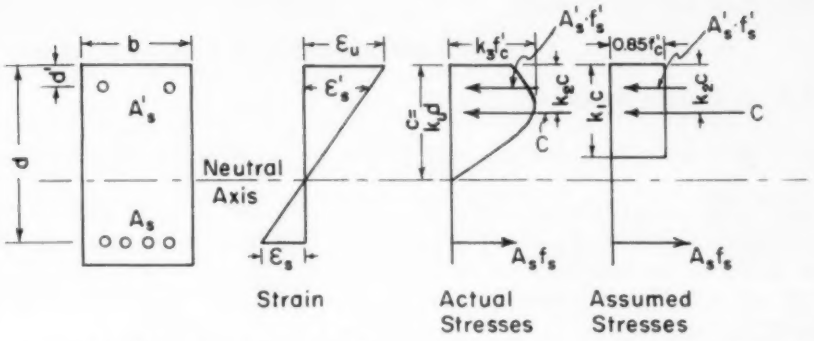

Fig. 12—Conditions at ultimate load, beam reinforced in tension and compression

will then be greater than the yield point strain ε_y', and the steel stress f_s' will equal f_y'. In this case we may rewrite Eq. (2.1) and (2.2) as follows:

$$0.85\, k_1 f_c'\, b\, c + A_s' f_y' = A_s f_y \qquad (2.1A)$$

$$M_{ult} = 0.85\, k_1 f_c'\, b\, c(d - k_2 c) + A_s' f_y'(d - d') \qquad (2.2A)$$

From Eq. (2.1A)

$$k_u = \frac{c}{d} = \left(\frac{f_y p - f_y' p'}{0.85\, k_1 f_c'}\right) \qquad (2.3)$$

or

$$\frac{c}{d} = \left(\frac{q - q'}{0.85\, k_1}\right) \qquad (2.3A)$$

where

$$q' = p' \frac{f_y'}{f_c'}$$

Calculate c from Eq. (2.3) and check that $f_s' = f_y'$ using

$$\varepsilon_s' = \varepsilon_u \frac{c - d'}{c} \qquad (2.4)$$

If $\varepsilon_s' \geq \varepsilon_y'$, the compression reinforcement has yielded and the ultimate moment may be calculated by solving Eq. (2.1A) and (2.2A), this yields:

$$M_{ult} = (A_s f_y - A_s' f_y')d \left\{1 - \frac{k_2}{0.85 k_1}\left(\frac{f_y p - f_y' p'}{f_c'}\right)\right\} + A_s' f_y' \{d - d'\}$$

Since $k_1 = 2k_2$, we may write:

$$M_{ult} = (A_s f_y - A_s' f_y')d \left\{1 - \frac{0.59}{f_c'}(f_y p - f_y' p')\right\} + A_s' f_y' \{d - d'\} \qquad (2.5)$$

This may also be written:

$$M_{ult} = (A_s f_y - A_s' f_y')d\{1 - 0.59(q - q')\} + A_s' f_y' \{d - d'\} \quad (2.5A)$$

If $f_y = f_y'$ this becomes

$$M_{ult} = (A_s - A_s')f_y d \left\{1 - 0.59 \frac{f_y}{f_c'}(p - p')\right\} + A_s' f_y \{d - d'\} \quad (2.5B)$$

This is Eq. A3 of the Appendix to ACI 318-56, and is valid only if the compression steel has yielded. When using this equation a check should be made on the compression steel stress using Eq. (2.3) and (2.4).

If $\varepsilon_s' < \varepsilon_y'$, then compatibility of strains in the compression zone must be considered.

$$f_s' = \varepsilon_s' E_s = \varepsilon_u E_s \left(\frac{c - d'}{c}\right) \quad (2.6)$$

Solving Eq. (2.1) and (2.6) for k_u:

$$k_u = \frac{c}{d} = \sqrt{\left[\frac{m}{2}\left(p' - \frac{\varepsilon_y}{\varepsilon_u} p\right)\right]^2 + p'm \frac{d'}{d}} - \frac{m}{2}\left(p' - \frac{\varepsilon_y}{\varepsilon_u} p\right) \quad (2.7)$$

where, as before, $m = (E_s \varepsilon_u)/(0.85 k_1 f_c')$.

The compression steel stress, f_s', may then be obtained from Eq. (2.6) by substituting the value of c, calculated using Eq. (2.7). The moment of resistance is obtained by substituting the calculated values of c and f_s' in Eq. (2.2).

Ultimate strength controlled by crushing of concrete—The tensile reinforcement is in this case stressed elastically. From equilibrium of internal forces:

$$0.85 k_1 f_c' b c + A_s' f_s' = A_s f_s \quad (2.8)$$

From equilibrium of internal and external moments:

$$M_{ult} = 0.85 k_1 f_c' b c (d - k_2 c) + A_s' f_s' (d - d') \quad (2.9)$$

If the compression reinforcement has yielded these equations become:

$$0.85 k_1 f_c' b c + A_s' f_y' = A_s f_s \quad (2.8A)$$

and

$$M_{ult} = 0.85 k_1 f_c' b c (d - k_2 c) + A_s' f_y' (d - d') \quad (2.9A)$$

From assumption of linear strain distribution:

$$k_u = \frac{c}{d} = \frac{\varepsilon_u}{\varepsilon_s + \varepsilon_u} \quad (2.10)$$

or

$$\varepsilon_s = \varepsilon_u \left(\frac{d - c}{c}\right) \quad (2.10A)$$

Solving Eq. (2.10) and (2.8A) for k_u

$$k_u = \frac{c}{d} = \sqrt{\left\{\frac{m}{2}\left(p + \frac{\varepsilon_y'}{\varepsilon_u}p'\right)\right\}^2 + pm} - \frac{m}{2}\left(p + \frac{\varepsilon_y'}{\varepsilon_u}p'\right) \quad (2.11)$$

where m is as before.

Check that ε_s' is $> \varepsilon_y'$ using $\varepsilon_s' = \varepsilon_u\,[(c - d')/c]$.

The moment of resistance is calculated using Eq. (2.9A), substituting the value of c, obtained from Eq. (2.11). If the compression reinforcement has not yielded, then the compatibility of strains across the entire section must be considered. This yields:

$$\varepsilon_s = \varepsilon_u\left(\frac{d - c}{c}\right) \quad (2.10A)$$

and

$$\varepsilon_s' = \varepsilon_u\left(\frac{c - d'}{c}\right) \quad (2.6)$$

Solving Eq. (2.10A), (2.6), and (2.8A) for k_u

$$k_u = \frac{c}{d} = \sqrt{\left\{\frac{m}{2}(p' + p)\right\}^2 + m\left(p'\frac{d'}{d} + p\right)} - \frac{m}{2}(p' + p) \quad (2.12)$$

The value of c obtained from Eq. (2.12) is substituted in Eq. (2.6) to give ε_s' and hence f_s'. These values of c and f_s' are substituted in Eq. (2.9) to yield the ultimate moment.

Balanced conditions—In this case simultaneous yielding of tensile steel and crushing of concrete takes place. From assumption of linear strain distribution:

$$c = \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u}\right)d \quad (2.13)$$

Solving Eq. (2.13) and (2.8):

Balanced steel ratio

$$p_b = p'\frac{f_s'}{f_y} + p_b \quad (2.14)$$

where p_b is the balanced steel ratio for a beam having the same steel and concrete strengths, but reinforced only in tension, as given by Eq. (1.10).

$$\varepsilon_s' = \varepsilon_u - \frac{d'}{d}(\varepsilon_y + \varepsilon_u)$$

If d'/d is $< (\varepsilon_u - \varepsilon_y')/(\varepsilon_u + \varepsilon_y)$, then $\varepsilon_s' > \varepsilon_y'$ and $f_s' = f_y'$

$$p_b = p'\frac{f_y'}{f_y} + p_b \quad (2.15)$$

If d'/d is $> (\varepsilon_u - \varepsilon_y')/(\varepsilon_u + \varepsilon_y)$, then $f_s' < f_y'$

$$p_s = p' \frac{E_s}{f_y} \left\{ \varepsilon_u - \frac{d'}{d} (\varepsilon_y + \varepsilon_u) \right\} + p_b \quad \text{(2.16)}$$

The ultimate moment at balanced failure is given by:

$$M_{ult} = 0.85\, k_1 f_c' bd^2 \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u} \right) \left\{ 1 - k_2 \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u} \right) \right\} + A_s' f_s' (d - d') \quad \text{(2.17)}$$

where

$$f_s' = f_y' \quad \text{if} \quad \frac{d'}{d} \leq \left(\frac{\varepsilon_u - \varepsilon_y'}{\varepsilon_u + \varepsilon_y} \right),$$

and

$$f_s' = E_s \left\{ \varepsilon_u - \frac{d'}{d} (\varepsilon_y + \varepsilon_u) \right\} \quad \text{if} \quad \frac{d'}{d} > \left(\frac{\varepsilon_u - \varepsilon_y'}{\varepsilon_u + \varepsilon_y} \right)$$

The results of applying the above analysis to 44 doubly reinforced beams is shown in detail in Table A-2 and is summarized below. (Histograms of these results are plotted in Fig. 13.)

Failure mode	Reinforcement	Average $\frac{M_{ult}\,(\text{test})}{M_{ult}\,(\text{calc})}$	Standard deviation
Tension	Structural or intermediate grade Steel	1.03	0.04
Tension	High strength alloy steel	1.21	0.17
Compression	Structural or intermediate grade Steel	1.03	0.07

It is of interest to note that the compression reinforcement yielded in only eight of the 33 beams failing in tension. In four of the beams the "compression" reinforcement was actually in tension at ultimate load.

The reason for the relatively high value of M_{test}/M_{calc} for beams failing in tension and reinforced with high strength alloy bars is probably strain hardening of the tension steel in beams with only a small percentage of tension reinforcement. The yield plateau of this reinforcement is of the order of 0.2 percent whereas that of structural grade reinforcement is about 1.5 percent. The alloy steel therefore quickly passes into the strain hardening range with consequently higher steel stresses at failure. Since neither the length of the yield plateau, nor the shape of the steel stress-strain curve beyond the yield plateau are included in steel specifications, it is the opinion of the writers that for practical design purposes the calculation of the ultimate moment of beams reinforced with high strength steel of this type should be based on the specified yield stress. However, if it is desired to analyze the

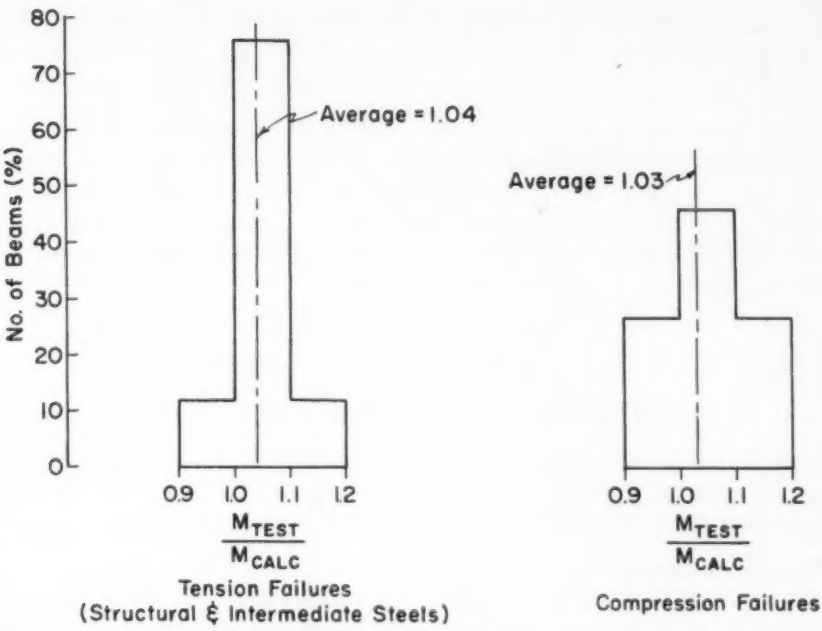

Fig. 13 — Histograms of (M_{test}/M_{calc}) for reinforced concrete beams with compression reinforcement

behavior of a specific beam, for which the stress-strain curve of the reinforcement is known, the following approach can be used.

From equilibrium of internal forces:

$$0.85\, k_1 f_c' \, b\, c + A_s' f_s' = A_s f_s \qquad (2.18)$$

From compatibility of strains:

$$\varepsilon_s' = \varepsilon_u \left(\frac{c - d'}{c} \right); \quad \text{and} \quad \frac{c}{d} = \left(\frac{\varepsilon_u}{\varepsilon_s + \varepsilon_u} \right)$$

Solving the above equations:

$$f_s = \left[\frac{0.85\, k_1 f_c'}{p} \left(\frac{\varepsilon_u}{\varepsilon_s + \varepsilon_u} \right) + \frac{p'}{p} E_s \varepsilon_u \left\{ 1 - \frac{d'}{d} \left(\frac{\varepsilon_s + \varepsilon_u}{\varepsilon_u} \right) \right\} \right] \qquad (2.19)$$

Plot the curve of f_s and ε_s given by Eq. (2.19) on the same base as the stress-strain curve for the reinforcement. The intersection of the curves gives the steel stress and strain at ultimate moment, from which the ultimate moment can be calculated. This approach was used to analyze Beam IIIB-1 from Table A-2, using a typical stress-strain curve for this type of reinforcement, and a value of M_{test}/M_{calc} of 1.23 was obtained, as against 1.44 if the calculated ultimate moment is based on the yield strength of the reinforcement. The remaining hyper-

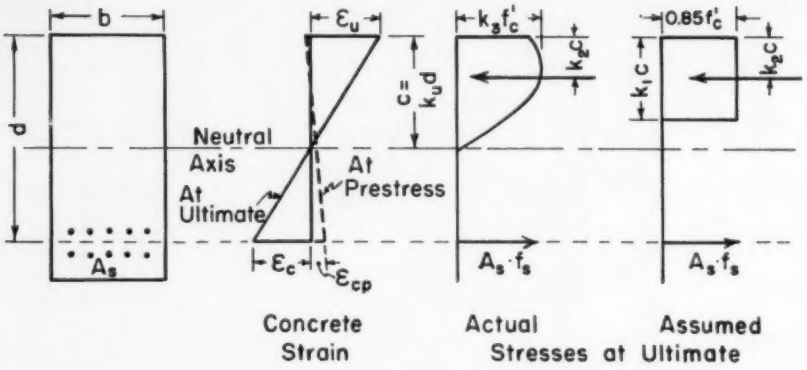

Fig. 14—Stress and strain conditions in prestressed beams

strength of 23 percent could be due to several as yet unpredictable causes such as variation in length of yield plateau from bar to bar, departure from linear strain relationship at a wide crack, etc.

Prestressed beams

The following reasoning applies to prestressed rectangular beams and to T- or I-section beams in which the flange thickness is greater than the depth of the equivalent rectangular stress block at failure.

From equilibrium of internal forces in Fig. 14:

$$0.85\, k_1 f_c'\, b\, c = A_s f_s \qquad (3.1)$$

From equilibrium of internal and external moments:

$$M_{ult} = A_s f_s (d - k_2 c) \qquad (3.2)$$

or

$$M_{ult} = 0.85\, k_1 f_c'\, b\, c\, (d - k_2 c) \qquad (3.2A)$$

From assumption of linear distribution of concrete strains:

$$k_u = \frac{c}{d} = \frac{\varepsilon_u}{\varepsilon_c + \varepsilon_u} \qquad (3.3)$$

Solving Eq. (3.1) and (3.3) we obtain:

(1) Change in concrete strain at level of steel from zero load to failure

$$(\varepsilon_c + \varepsilon_{cp}) = \varepsilon_{cp} + \varepsilon_u \left(\frac{0.85\, k_1 f_c'}{p f_s} - 1 \right) \qquad (3.4)$$

Because of high bond stresses and consequent bond slip at failure in this type of beam, the change in strain in the steel during loading to failure may not be equal to the change in strain in the adjacent concrete. We will write therefore:

(2) Change in strain in steel during loading to failure:

$$= \varepsilon_s - \varepsilon_{se} = F \left\{ \varepsilon_{cp} + \varepsilon_u \left(\frac{0.85\, k_1 f_c'}{p f_s} - 1 \right) \right\} \quad \text{.....(3.5)}$$

where F is the ratio of the actual change in strain of the steel, to the change in strain of the concrete at the level of the steel calculated on the basis of linear distribution of concrete strain.

It has been suggested[23] that for well bonded tendons $F = 1.0$, while for post-tensioned tendons without bond a suitable value deduced from the experimental results[10,24,25,26] plotted in Fig. 15 is $F = 0.8k_u$. Therefore, for prestressed beams with well bonded tendons:

$$\varepsilon_s = \left\{ \varepsilon_u \left(\frac{0.85\, k_1 f_c'}{p f_s} - 1 \right) + \varepsilon_{cp} + \varepsilon_{se} \right\} \quad \text{.....(3.6)}$$

while for prestessed beams with unbonded tendons:

$$\varepsilon_s = \left\{ \frac{0.8 p f_s}{0.85\, k_1 f_c'} (\varepsilon_{cp} - \varepsilon_u) + 0.8\, \varepsilon_u + \varepsilon_{se} \right\} \quad \text{.....(3.7)}$$

Since the shapes of the stress-strain curves of prestressing steel do not in general lend themselves to algebraic representation, the analysis of a given section is best carried out using a process of iteration. Assuming a value of f_s, the steel stress at failure, the corresponding strain ε_s may be calculated using Eq. (3.6) or (3.7). The steel strain so obtained is entered on the stress-strain diagram for the reinforcing steel and a new value of f_s obtained. The process is repeated, using the new value of f_s, as often as is necessary.

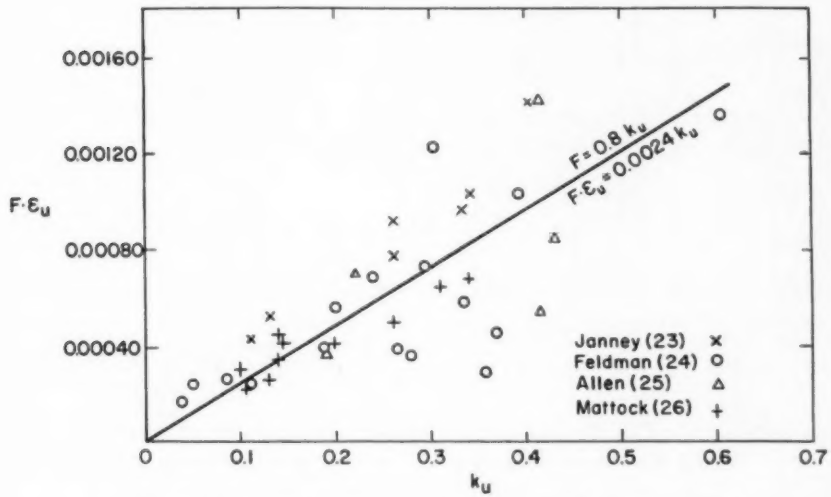

Fig. 15—Post-tensioned prestressed beams without bond $F \cdot \varepsilon_u$ v k_u

When the steel stress has been determined, the neutral axis depth may be calculated using Eq. (3.1) and the ultimate moment using Eq. (3.2).

Value of ε_{cp}: For a safe estimate of ε_s, the minimum probable value of ε_{cp} should be used. It can be shown[26] that with present day allowable concrete stresses a reasonable value of ε_{cp} is 0.0004.

The analysis proposed above was applied to 32 prestressed beams with well bonded tendons and to 24 post-tensioned beams without bond; the results are detailed in Tables A-4 and A-5 and are summarized below.

1. Beams with well bonded tendons:
   Average M_{test}/M_{calc} = 1.07; Standard deviation = 0.077
2. Post-tensioned beams without bond:
   Average M_{test}/M_{calc} = 1.05; Standard deviation = 0.094

Histograms of these results are plotted in Fig. 16.

Note — Expressions for the strength of prestressed beams with non-rectangular compression zones may be derived by combining the approach used above to take into account the influence of prestress, with the approach described below for reinforced concrete beams with nonrectangular compression zones.

NONRECTANGULAR BEAMS

Symmetrical T-beams

Ultimate strength controlled by yielding of reinforcement — When the flange thickness exceeds the depth of the equivalent rectangular stress block, the equations developed for the singly reinforced rectangu-

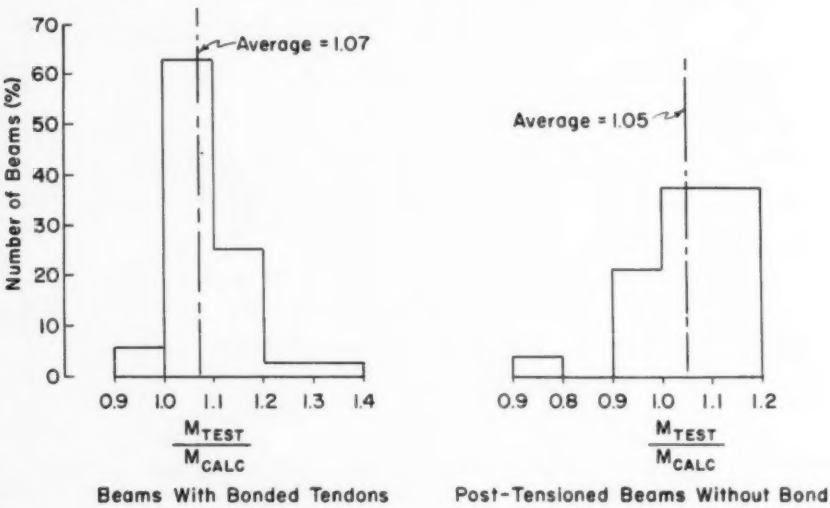

Fig. 16 — Histograms of (M_{test}/M_{calc}) for prestressed beams with and without bond

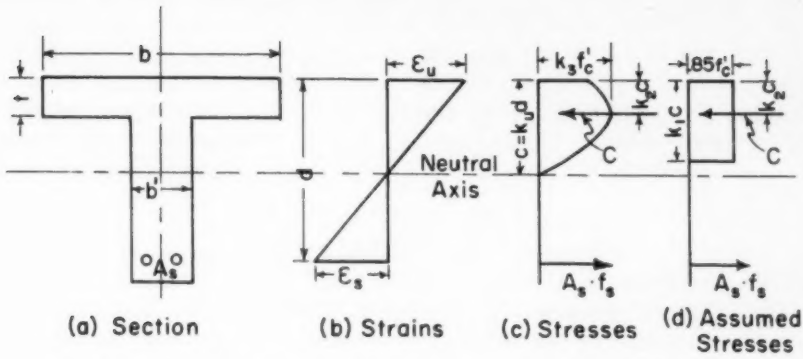

Fig. 17—Conditions at ultimate strength in reinforced concrete T-beam

lar beams are valid for the T-beam. If the depth of the equivalent rectangular stress block is greater than the flange thickness, i.e., if

$k_1 c = (A_s f_y)/(0.85 f_c' b) > t$ (Fig. 17), then:

From equilibrium of internal forces:

$$0.85 k_1 f_c' b' c + 0.85 f_c' (b - b') t = A_s f_y \quad \text{.....(4.1)}$$

From equilibrium of internal and external moments:

$$M_{ult} = 0.85 k_1 f_c' b' c \left(d - \frac{k_1}{2} c \right) + 0.85 f_c' (b - b') t \left(d - \frac{t}{2} \right) \quad \text{.....(4.2)}$$

Solving Eq. (4.1) and (4.2):

$$M_{ult} = (A_s - A_{sf}) f_y d \left\{ 1 - 0.59 \frac{f_y}{b'd f_c'} (A_s - A_{sf}) \right\} + A_{sf} f_y \left(d - \frac{t}{2} \right) \quad (4.3)$$

In which $A_{sf} = \{0.85 f_c' (b - b') t\}/f_y$ is the steel area necessary to develop the compressive strength of the overhanging portions of the flange. Eq. (4.3) may also be written as follows:

$$M_{ult} = (A_s - A_{sf}) f_y d \left\{ 1 - 0.59 (p_w - p_f) \frac{f_y}{f_c'} \right\} + A_{sf} f_y \left(d - \frac{t}{2} \right) \quad \text{.....(4.4)}$$

where: $p_w = A_s/b'd$ and $p_f = A_{sf}/b'd$. This is Eq. (A4) of ACI 318-56.

Ultimate strength controlled by crushing of concrete—While steel stress is still below the yield point:

From equilibruim of internal forces:

$$0.85 k_1 f_c' b' c + 0.85 f_c' (b - b') t = A_s f_s \quad \text{.....(4.1A)}$$

From equilibrium of internal and external moments:

$$M_{ult} = 0.85 k_1 f_c' b' c \left(d - \frac{k_1}{2} c \right) + 0.85 f_c' (b - b') t \left(d - \frac{t}{2} \right) \quad \text{.....(4.2)}$$

From assumption of linear strain distribution:

$$k_u = \frac{c}{d} = \frac{\varepsilon_u}{\varepsilon_s + \varepsilon_u} \quad (4.5)$$

Solving Eq. (4.1A) and (4.5) for k_u:

$$k_u = \frac{c}{d} = \sqrt{p_w m + \left[\left\{\left(\frac{b-b'}{b'}\right)\frac{t}{2k_1 d}\right\} + \frac{p_w m}{2}\right]^2} - \left[\left\{\left(\frac{b-b'}{b'}\right)\frac{t}{2k_1 d}\right\} + \frac{p_w m}{2}\right] \quad (4.6)$$

where, as before, $m = (E_s \varepsilon_u)/(0.85 k_1 f_c')$.

The ultimate moment of resistance of the section may be calculated by substituting the value of c obtained from Eq. (4.6) in Eq. (4.2) above.

Balanced conditions—i.e., simultaneous yielding of steel and crushing of concrete.

From assumption of linear strain distribution:

$$\frac{c}{d} = \left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) \quad (4.7)$$

Solving Eq. (4.1) and (4.7):

$$\left(\frac{A_s - A_{sf}}{b'd}\right) = 0.85 k_1 \frac{f_c'}{f_y}\left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u}\right) = p_b \quad (4.8)$$

Where p_b is the balanced steel ratio for a rectangular beam made of same concrete and steel.

The ultimate moment for balanced conditions may be written:

$$M_{ub} = 0.85 k_1 f_c' b' \left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) d^2 \left\{1 - \frac{k_1}{2}\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right)\right\} + 0.85 f_c' (b - b') t \left(d - \frac{t}{2}\right) \quad (4.9)$$

The results of tests on 15 T-beams were analyzed and are detailed in Table A-3. The mean value of M_{test}/M_{calc} is 1.11 and the standard deviation 0.10. The results include some beams in which strain hardening of the reinforcement is known to have occurred. If desired the strain hardening effect could be investigated in a particular case using the approach outlined for doubly reinforced beams.

Nonsymmetrical T-beams

For most nonsymmetrical T-beams used in monolithically cast construction, lateral deflection is prevented. The neutral axis is then horizontal, and the strength of an unsymmetrical T-beam equals that of an equivalent symmetrical beam with the same total flange width.

When lateral restraint is absent, however, the beam may be considered to be loaded vertically and to be free to deflect laterally as shown in Fig. 18. Since no moments are applied in a horizontal plane, the

center of concrete compression must be vertically above the centroid of the tension reinforcement. The width of the stress block at the top face of the beam will therefore be 1:5 times the width of the web of the beam for this condition to be satisfied, assuming the reinforcement is placed symmetrically within the web.

Ultimate strength controlled by yield of reinforcement—From equilibrium of internal forces:

$$\tfrac{1}{2}\ (1.5\ b'\ a\ 0.85\ f_c') = A_s\ f_y \qquad (5.1)$$

From equilibrium of internal and external moments:

$$M_{ult} = A_s\ f_y \left(d - \frac{a}{3} \right) \qquad (5.2)$$

or

$$= 0.75\ b'\ a\ 0.85\ f_c' \left(d - \frac{a}{3} \right) \qquad (5.2A)$$

Solving Eq. (5.1) and (5.2) we have:

$$M_{ult} = A_s\ f_y\ d\ (1 - 0.52\ q_w) \qquad (5.3)$$

or

$$M_{ult} = b'\ d^2\ f_c'\ q_w\ (1 - 0.52\ q_w) \qquad (5.3A)$$

where

$$q_w = \left(\frac{A_s}{b'\ d}\ \frac{f_y}{f_c'} \right)$$

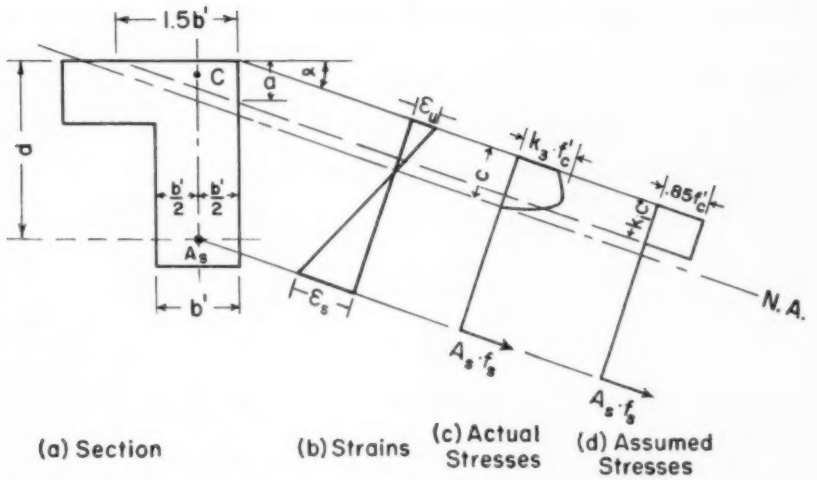

Fig. 18—Conditions at ultimate strength in a reinforced concrete unsymmetrical T-beam

Ultimate strength controlled by crushing of concrete—While the steel stress is below the yield point.

From the geometry of the section, as shown in Fig. 18:

$$c = \frac{3}{2\,k_1} b' \sin \alpha \qquad (5.4)$$

where α is the inclination of the neutral axis to the horizontal.

From the assumption of linear strain distribution:

$$\frac{\varepsilon_s}{\varepsilon_u} = \frac{(d \cos \alpha + \tfrac{1}{2} b' \sin \alpha - c)}{c} \qquad (5.5)$$

From equilibrium of internal forces:

$$\tfrac{1}{2} (\tfrac{3}{2} b')^2 \tan \alpha \, 0.85 f_c' = A_s E_s \varepsilon_s \qquad (5.6)$$

From equilibrium of internal and external moments:

$$M_{ult} = \tfrac{1}{2} (\tfrac{3}{2} b')^2 \tan \alpha \, 0.85 f_c' (d - \tfrac{1}{2} b' \tan \alpha) \qquad (5.7)$$

Solving Eq. (5.4), (5.5), and (5.6) for tan α we have:

$$\tan \alpha = \left[\frac{\sqrt{\left(1 - \frac{k_1}{3}\right)^2 + \left(\frac{3\,b'\,d}{m\,A_s}\right)} - \left(1 - \frac{k_1}{3}\right)}{\left(\frac{9\,(b')^2}{4\,k_1\,m\,A_s}\right)} \right] \qquad (5.8)$$

where, as before, $m = (\varepsilon_u E_s) / (0.85 k_1 f_c')$.

The ultimate moment is calculated by substituting the value of tan α from Eq. (5.8) in Eq. (5.7).

Balanced condition—Simultaneous crushing of concrete and yield of steel.

From equilibrium of internal forces:

$$\tfrac{1}{2} (\tfrac{3}{2} b')^2 \tan \alpha \, 0.85 f_c' = A_s f_y = A_s E_s \varepsilon_y \qquad (5.9)$$

From assumption of linear strain distribution:

$$\frac{\varepsilon_y}{\varepsilon_u} = \frac{(d \cos \alpha + \tfrac{1}{2} b' \sin \alpha) - \left(\frac{3}{2\,k_1} b' \sin \alpha\right)}{\left(\frac{3}{2\,k_1} b' \sin \alpha\right)}$$

$$\therefore \frac{\varepsilon_y}{\varepsilon_u} = \frac{2\,k_1}{3}\left(\frac{d}{b'} \cot \alpha + \tfrac{1}{2}\right) - 1 \qquad (5.10)$$

Solving Eq. (5.9) and (5.10) for the steel ratio p_{wb} for balanced ultimate conditions:

$$p_{wb} = \frac{f_c'}{f_y}\left[\frac{1.91\,k_1}{3\left(\frac{\varepsilon_y}{\varepsilon_u} + 1\right) - k_1} \right] \qquad (5.11)$$

or

$$q_{wb} = \frac{1.91\, k_1}{3\left(\frac{\varepsilon_y}{\varepsilon_u} + 1\right) - k_1} \quad \text{(5.11A)}$$

The ultimate moment of resistance is given by substituting q_{wb} in Eq. (5.3)

$$M_{ult} = A_s f_y d\,(1 - 0.52\, q_{wb}) \quad \text{(5.12)}$$

The results of tests on 12 unsymmetric T-beams are summarized in Table A-6. The mean value of M_{test}/M_{calc} for all beams is 1.10, and the standard deviation is 0.138. The mean value of M_{test}/M_{calc} for beams with ultimate strength controlled by tension is 1.04 and the standard deviation is 0.047.

Beams with triangular compression zones

The following analysis was developed in connection with tests of this type of beam, to check the applicability of the rectangular stress block approximation to members having triangular shaped compression zones.

Ultimate strength controlled by yield of reinforcement—From equilibrium of internal forces (see Fig. 19)

$$0.85\, f_c' a^2 \tan\theta = A_s f_y \quad \text{(6.1)}$$

From equilibrium of internal and external moments:

$$M_{ult} = A_s f_y \left(d - \frac{2a}{3}\right) \quad \text{(6.2)}$$

Solving Eq. (6.1) and (6.2) we have:

$$M_{ult} = A_s f_y d\,(1 - 0.723\sqrt{q}) \quad \text{(6.3)}$$

or

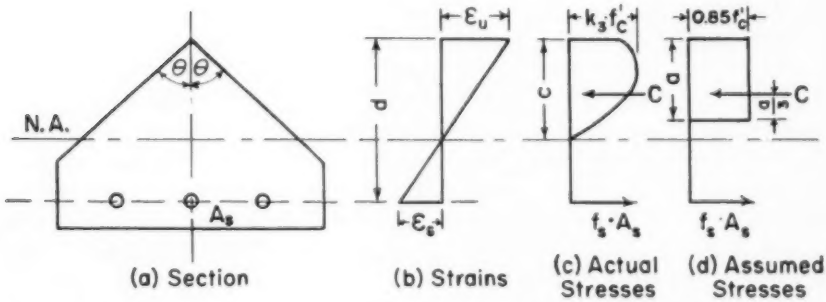

Fig. 19—Condition at ultimate strength in a reinforced concrete beam with a triangular shaped compression zone

$$M_{ult} = d^3 \tan \theta \, f_c' \, q \, (1 - 0.723 \sqrt{q}) \quad \text{(6.3A)}$$

where

$$q = \left(\frac{A_s}{d^2 \tan \theta} \frac{f_y}{f_c'} \right)$$

Ultimate strength controlled by crushing of the concrete—While the steel stress is below the yield point.

From equilibrium of internal forces:

$$0.85 \, f_c' \, a^2 \tan \theta = A_s f_s \quad \text{(6.4)}$$

From equilibrium of internal and external moments:

$$M_{ult} = 0.85 \, f_c' \, a^2 \tan \theta \left(d - \frac{2a}{3} \right) \quad \text{(6.5)}$$

From assumption of linear strain distribution:

$$\frac{\varepsilon_s}{\varepsilon_u} = \left(\frac{d - c}{c} \right) \quad \text{(6.6)}$$

From Eq. (6.4) and (6.6):

$$c^3 \left(\frac{0.85 \, f_c' \, k_1^3 \tan \theta}{A_s E_s \varepsilon_u} \right) + c - d = 0 \quad \text{(6.7)}$$

Solve Eq. (6.7) for c and substitute in Eq. (6.5A) below to compute ultimate moment

$$M_{ult} = 0.85 \, f_c' \, k_1^2 c^3 \tan \theta \left(d - \frac{2k_1}{3} c \right) \quad \text{(6.5A)}$$

Alternatively Eq. (6.4) and (6.6) can be solved for c using an iterative procedure. The value of c obtained is then, as before, substituted in Eq. (6.5A).

Balanced conditions—Simultaneous crushing of concrete and yielding of steel.

From equilibrium of internal forces:

$$0.85 \, f_c' \, (k_1 c)^2 \tan \theta = A_s f_y \quad \text{(6.8)}$$

From assumption of linear strain distribution:

$$\frac{c}{d} = \left(\frac{\varepsilon_u}{\varepsilon_y + \varepsilon_u} \right) \quad \text{(6.9)}$$

Solving Eq. (6.8) and (6.9) for the steel ratio p_b for balanced ultimate conditions [$p = A_s/(d^2 \tan \theta)$].

$$p_b = \frac{0.85 \, f_c'}{f_y} \left(\frac{k_1 \varepsilon_u}{\varepsilon_y + \varepsilon_u} \right)^2 \quad \text{(6.10)}$$

or

$$q_b = 0.85 \left(\frac{k_1 \varepsilon_u}{\varepsilon_y + \varepsilon_u} \right)^2 \quad \text{(6.10A)}$$

The ultimate moment of resistance is given by substituting q_b in Eq. (6.3)

$$M_{ult} = A_s f_y d \, (1 - 0.723 \sqrt{q_b}) \quad \text{(6.11)}$$

The results of tests on six beams with triangular compression zone are summarized in Table A-7. The mean value of M_{test}/M_{calc} is 1.10 and the standard deviation is 0.069.

DESIGN OF COLUMNS

Concentrically loaded columns

In this case it is assumed that at ultimate strength of a column the entire concrete cross-section is uniformly stressed to 85 percent of the cylinder strength of the concrete, and that the entire cross section of the column is uniformly strained to 0.003 in. per in.

For columns reinforced with steels having a yield point of 90,000 psi or less, ultimate strength will be controlled by simultaneous crushing of the concrete and yielding of the steel. The ultimate strength is therefore given by:

$$P_o = 0.85 f_c' A_c + f_y A_{st} \quad \text{(7.1)}$$

where A_c is the net concrete cross section
A_{st} is the total longitudinal reinforcement cross section

In the above it is assumed that sufficient lateral ties are provided to ensure that the reinforcement will not buckle before reaching its yield stress.

Eq. (7.1) was used to calculate the ultimate strength of 16 concentrically loaded reinforced concrete columns with lateral ties, tested as part of the ACI column investigation and reported in University of Illinois, *Engineering Experiment Station Bulletin* No. 267.[6] The results are set out in detail in Table A-8. The average value of P_{test}/P_{calc} for all beams is 1.00, and the standard deviation is 0.074.

It was concluded in the ACI column investigation[6] that Eq. (7.1) also serves to calculate the yield point load of a concentrically loaded circular column with spiral reinforcement. Further increase in load is obtainable in this type of column when loaded concentrically due to the lateral restraint of the compressed concrete by the spiral. However, Hognestad[8] has shown that if the load is eccentric to even a small degree, then no further increase in load is obtained after yield in this type of column. Since a truly concentrically loaded column is an extreme rarity in practice, it is proposed that increase in load after yield of

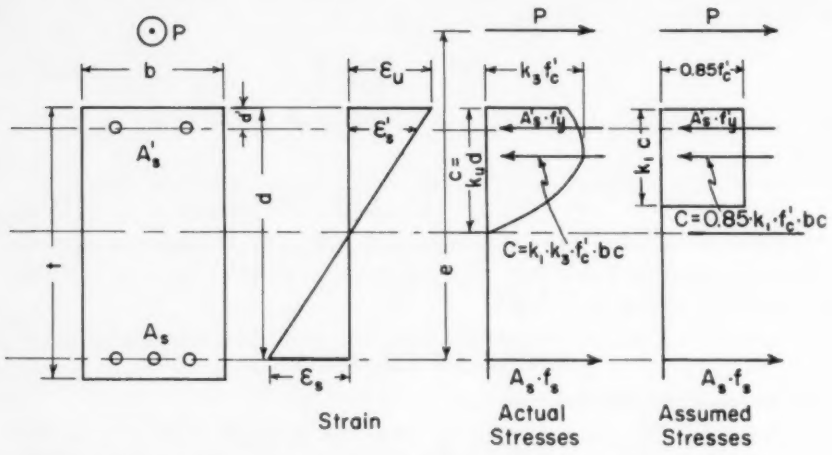

Figure 20—Conditions in an eccentrically loaded column at failure, $c < d$

spirally reinforced columns should be ignored, and that their ultimate strength should be calculated using Eq. (7.1).

Eccentrically loaded rectangular column, reinforced on two faces

It is reasonable to assume that the compression reinforcement has yielded. This may be verified by checking that $\varepsilon_u = [(c - d')/c]$ is $\geq \varepsilon_y'$. If this is not the case, a solution may be developed by taking into account compatibility of strains, as was done for the case of a beam reinforced in compression in which the compression reinforcement did not yield.

In what follows it is assumed that the above check has been made, and that the compression reinforcement has been found to yield.

From equilibrium of internal and external forces in Fig. 20:

$$P_u = 0.85 k_1 f_c' b c + A_s' f_y' - A_s f_s \quad \text{(8.1)}$$

From equilibrium of internal and external moments:

$$P_u e = 0.85 k_1 f_c' b c (d - k_3 c) + A_s' f_y' (d - d') \quad \text{(8.2)}$$

Strength governed by tension steel yielding—Solving Eq. (8.1) and (8.2) for c/d, and substituting

$$q = \frac{p f_y}{f_c'} \quad \text{and} \quad q' = \frac{p' f_y'}{f_c'}$$

$$\frac{c}{d} = \frac{1}{k_1}\left[- \left(\frac{e}{d} - 1\right) + \sqrt{\left\{\frac{e}{d} - 1\right\}^2 - 2.35 \left\{ q' \left(\frac{e + d'}{d} - 1\right) - q \left(\frac{e}{d}\right)\right\}}\right] \quad \text{(8.3)}$$

Substituting this value of c/d in Eq. (8.1) yields:

$$P_u = f_c' b d \left[(q' - q) + 0.85 \left\{ -\left(\frac{e}{d} - 1\right) + \sqrt{\left(\frac{e}{d} - 1\right)^2 - 2.35\ q' \left(\frac{e + d'}{d} - 1\right) - q\ \frac{e}{d}} \right\} \right] \quad \text{.....(8.4)}$$

For symmetrically reinforced columns, $p = p'$. Assuming $f_y = f_y'$, $q = q'$, therefore:

$$P_u = 0.85 f_c' b d \left[-\left(\frac{e}{d} - 1\right) + \sqrt{\left(\frac{e}{d} - 1\right)^2 + 2.35\, q \left(\frac{d - d'}{d}\right)} \right] \quad \text{....(8.5)}$$

In the above equations the displacement of concrete by the compression reinforcement is ignored. For values of $p' > 2.0$ percent the ultimate load may be over-estimated by up to 6 percent.[8] This error may be corrected by considering an effective yield point $[f_y' - 0.85 f_c']$ instead of f_y' in the calculation q'.

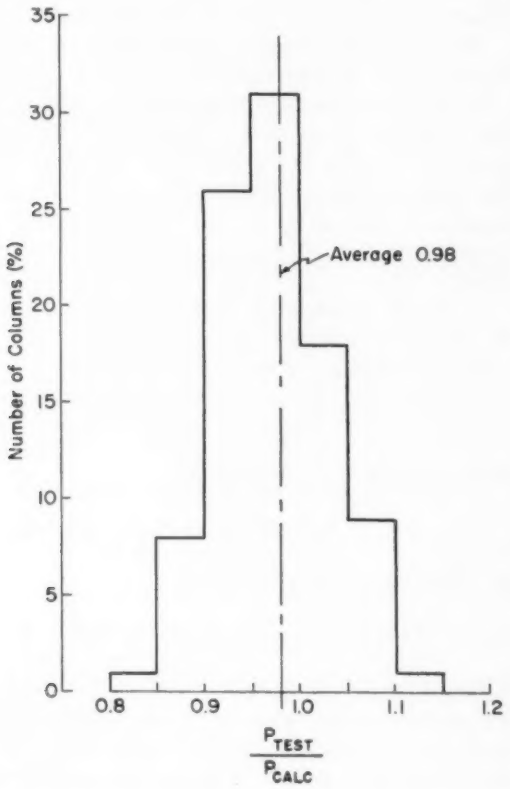

Fig. 21 — Histogram of (P_{test}/P_{calc}) for eccentrically loaded rectangular columns

Strength governed by concrete crushing—From assumption of linear distribution of strains:

$$\frac{c}{d} = \left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_s}\right) \quad (8.6)$$

and,

$$f_s = E_s\,\varepsilon_s = E_s\,\varepsilon_u \left(\frac{d-c}{c}\right) \quad (8.6A)$$

If $k_1\,c$ is less than t, Eq. (8.1) and (8.2) apply.

If $k_1\,c$ is greater than t, these equations become:

$$P_u = 0.85\,f_c'\,b\,t + A_s'\,f_y' + A_s\,f_s \quad (8.1A)$$

$$P_u e = 0.85\,f_c'\,b\,t\left(d - \frac{t}{2}\right) + A_s'\,f_y'\,(d-d') \quad (8.2A)$$

Once again, if greater precision is desired in computation of the ultimate load, an effective yield point $[f_y' - 0.85\,f_c']$ should be used instead of f_y'.

Solving Eq. (8.1), (8.2), and (8.6A) yields a cubic equation in c/d which does not lend itself to easy solution.

The following method of solution by successive approximations was proposed by Hognestad.[8]

With known section dimensions, material constants, and eccentricity e, the problem is to find c, and hence P_u. The procedure is as follows. Assume a value for c and use Eq. (8.2) to calculate P_u. Substituting the assumed value for c in Eq. (8.6A) calculate f_s. Using the calculated values of P_u and f_s in Eq. (8.1) calculate c. The process is repeated until the assumed and final value of "c" coincide or are sufficiently close.

Balanced conditions — Simultaneous yielding of tension steel and crushing of concrete.

In this case $\varepsilon_s = \varepsilon_y$

$$\therefore \frac{c}{d} = \left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) \quad (8.7)$$

Solving Eq. (8.1) and (8.7):

$$P_u = 0.85\,k_1\,f_c'\,b\,d\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) + A_s'\,f_y' - A_s\,f_y \quad (8.8)$$

Solving Eq. (8.2) and (8.7):

$$P_u e = 0.85\,k_1\,f_c'\,b\,d^2\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right)\left\{1 - k_2\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right)\right\} + A_s'\,f_y\,(d-d') \quad (8.9)$$

The analysis proposed above was applied to 84 eccentrically loaded, rectangular reinforced concrete columns tested by Hognestad.[8] The results are set out in detail in Table A-9. The average value of P_{test}/P_{calc} for all columns is 0.97, and the standard deviation is 0.059. A histogram

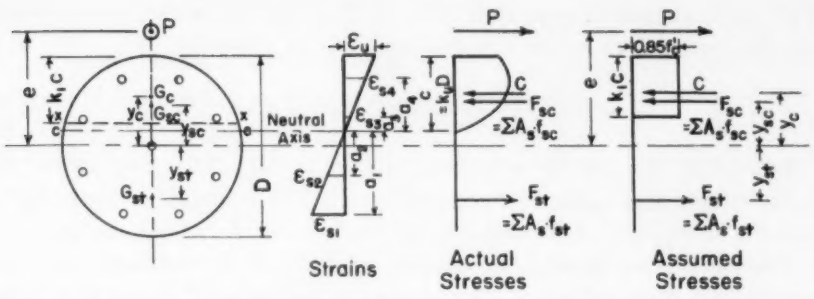

Fig. 22—Stress and strain conditions in circular column subject to eccentric loading

of these results is plotted in Fig. 21. The calculation of the ultimate strength of this large number of columns was greatly facilitated by use of an electronic computer.

Circular columns subject to eccentric load

Case 1—Equivalent rectangular stress distribution covers part only of column cross section as shown in Fig. 22.

Let G_{sc} be center of action of steel compressive force.

Let G_{st} be center of action of steel tensile force.

Let G_c be center of action of concrete compressive force.

From condition of equilibrium of internal and external forces:

$$P_u = 0.85 f_c' A_c + F_{sc} - F_{st} \quad (9.1)$$

where

A_c = area of segment of circle covered by equivalent rectangular stress distribution.

F_{sc} = resultant steel compressive force ($\Sigma A_s' f_{sc}$)

F_{st} = resultant steel tensile force ($\Sigma A_s f_{st}$)

From condition of equilibrium of internal and external moments:

$$P_u e = 0.85 f_c' A_c y_c + F_{sc} y_{sc} + F_{st} y_{st} \quad (9.2)$$

Where the eccentricity e is measured from the center of the column.

For the general case an iterative solution of the Eq. (9.1) and (9.2) is proposed.

Assume a position of the neutral axis cc and calculate the stress in each bar of the reinforcement using:

$$f_{sn} = E_s \varepsilon_{sn} = E_s \varepsilon_u \left(\frac{a_n}{c}\right) \text{ ; which must be } < f_y$$

Calculate the resultant tensile and compressive steel forces, F_{st} and F_{sc}, and their centers of action, $G_{st\,(y_{st})}$; $G_{sc(y_{sc})}$.

The first moment of area about center O of the segmental area covered by the rectangular stress distribution, $A_c y_c$, is given by:

$$A_c y_c = \frac{(\overline{xx})^3}{12} = \tfrac{2}{3} \{ D (k_1 c) - (k_1 c)^2 \}^{3/2} \quad \text{.....(9.3)}$$

Substitute the values calculated above in Eq. (9.2) and obtain P_u.

Substitute this value of P_u together with the calculated values of F_{sc} and F_{st} in Eq. (9.1) to obtain A_c, and hence a value for $k_1 c$ and c.

The calculation should be repeated as often as is necessary, until assumed and final values of c are equal or sufficiently close. The value of P_u corresponding to this value of c is the ultimate load capacity of the column.

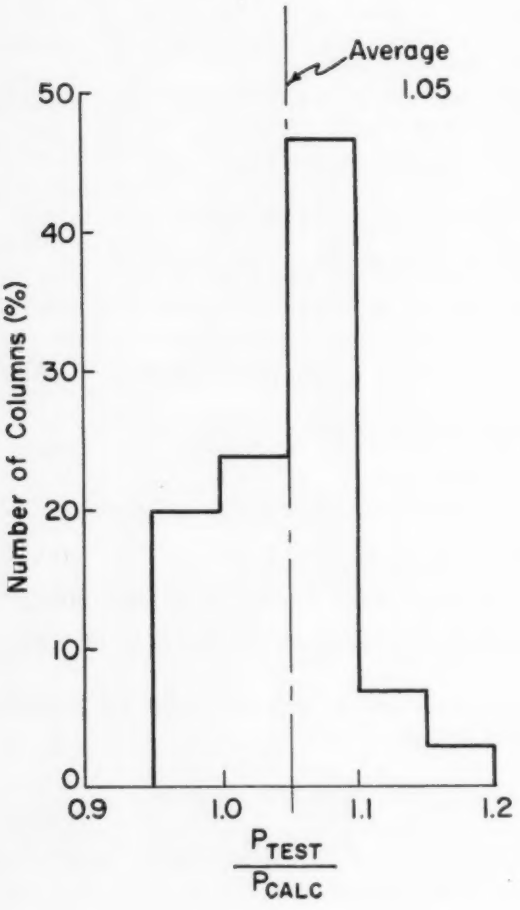

Fig. 23 — Histogram of (P_{test}/P_{calc}) for 30 eccentrically loaded circular columns

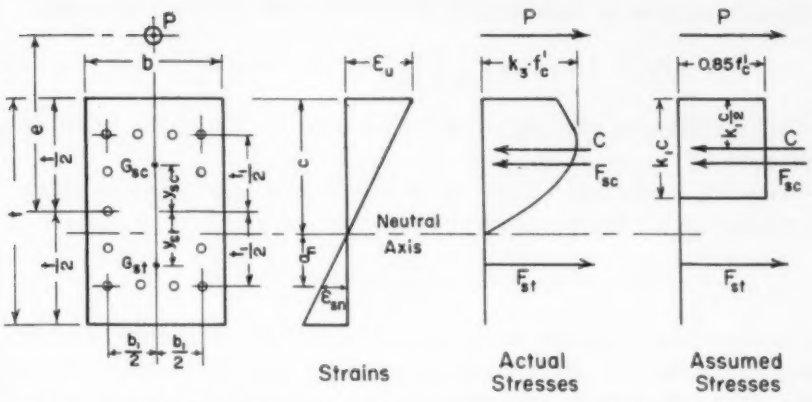

Fig. 24—Conditions at ultimate load, eccentrically loaded rectangular column with reinforcement on all four faces

Case 2—Equivalent rectangular stress distribution covers the entire column cross section.

From condition of equilibrium of internal and external forces:

$$P_u = 0.85 f_c' \left(\frac{\pi}{4} D^2\right) + F_{sc} \quad \text{.....(9.4)}$$

From condition of equilibrium of internal and external moments:

$$P_u e = F_{sc}\, y_{sc} \quad \text{.....(9.5)}$$

As before assume a position for the neutral axis and calculate F_{sc} and y_{sc}. Using these values of F_{sc} and y_{sc} in Eq. (9.5) calculate P_u. From Eq. (9.4) calculate F_{sc} inserting value of P_u found above.

Adjust the assumed neutral axis position until the two values of F_{sc} calculated are equal or sufficiently close.

The above method of analysis was applied to 30 eccentrically loaded circular columns tested by Hognestad[8] and the results are contained in Table A-10. The average of P_{test}/P_{calc} for the whole series was 1.05, and the standard deviation 0.060. A histogram of the results is plotted in Fig. 23.

Rectangular columns, reinforced at all four faces (Fig. 24)

Let G_{sc} be center of action of steel compressive force.

Let G_{st} be center of action of steel tensile force.

Let F_{sc} be resultant steel compressive force ($\Sigma A_s f_{sc}$).

Let F_{st} be resultant steel tensile force ($\Sigma A_s f_{st}$).

From condition of equilibrium of internal and external forces:

$$P_u = 0.85 f_c' b k_1 c + F_{sc} - F_{st} \quad \text{.....(10.1)}$$

From condition of equilibrium of internal and external moments:

$$P_u e = 0.85 f_c' b k_1 c \left(\frac{t}{2} - k_1 \frac{c}{2}\right) + F_{sc} y_{sc} + F_{st} y_{st} \quad \text{.....(10.2)}$$

In the general case an iterative solution of the above equations is proposed.

Assume a position of the neutral axis and calculate the stress in each bar of the reinforcement using:

$$f_{sn} = E_s \varepsilon_{sn} = E_s \varepsilon_u \left(\frac{a_n}{c}\right) ; \text{ which must be } \leq f_y$$

Calculate the resultant steel forces F_{sc} and F_{st}, and their centers of action G_{sc} and G_{st}. With these values of F_{sc} and F_{st}, calculate P_u using Eq. (10.1). Substitute this value of P_u in Eq. (10.2) together with the previously calculated values of F_{sc}, F_{st}, y_{sc}, y_{st}, and hence calculate the neutral axis depth c.

The calculation should be repeated as often as is necessary until the assumed and final values of c coincide or are sufficiently close. The value of P_u corresponding to this value of c is the ultimate load capacity of the column for the particular eccentricity considered.

Rectangular columns with bending on both principal axes (Fig. 25)

Let G_{sc} be center of action of steel compressive force.
Let G_{st} be center of action of steel tensile force.
Let G_c be center of action of concrete compressive force.

From condition of equilibrium of internal and external forces:

$$P_u = 0.85 f_c' A_c + F_{sc} - F_{st} \quad \text{.....(11.1)}$$

where

A_c = area of compression zone covered by rectangular stress block
F_{sc} = resultant steel compressive force ($\Sigma A_s f_{sc}$)
F_{st} = resultant steel tensile force ($\Sigma A_s f_{st}$)

From condition of equilibrium of internal and external moments:

$$0.85 f_c' A_c x_c = (F_{st} x_{st} - F_{sc} x_{sc} - P_u e_x) \quad \text{.....(11.2)}$$

$$0.85 f_c' A_c y_c = (F_{st} y_{st} - F_{sc} y_{sc} - P_u e_y) \quad \text{.....(11.3)}$$

In the general case an iterative solution of the above three equations is proposed.

Assume a position of the neutral axis cc and calculate the stress in each bar of the reinforcement using:

$$f_{sn} = E_s \varepsilon_{sn} = E_s \varepsilon_u \left(\frac{a_n}{c}\right) ; \text{ which must be } \leq f_y$$

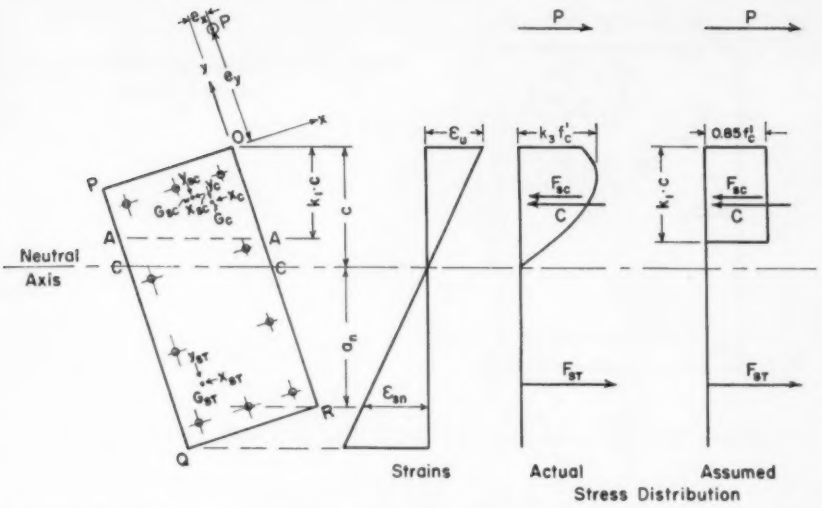

Fig. 25—Conditions at ultimate load in column subject to axial load and bend-ind on both principal axes

Calculate the resultant tensile and compressive steel forces, F_{st} and F_{sc}, and their centers of action $G_{st}(x_{st}, y_{st})$ and $G_{sc}(x_{sc}, y_{sc})$. Using the above values of F_{st} and F_{sc}, find P_u for the assumed neutral axis position using Eq. (11.1). From Eq. (11.2) and (11.3) obtain $A_c y_c$ and $A_c x_c$, using the values of P_u, F_{sc}, etc. already found.

It is then possible to calculate the dimensions of the area covered by the rectangular stress distribution and hence to obtain the position of the neutral axis. The calculation must be repeated as often as is necessary until assumed and calculated positions of neutral axis coincide or are sufficiently close. The value of P_u corresponding to this neutral axis position is the ultimate strength of the column.

The calculation of the dimensions of the compression zone from values of $A_c y_c$ and $A_c x_c$ obtained as outlined above is considered below.

Case 1—Line AA cuts adjacent sides OP and OR as in Fig. 26.

$$A_c x_c = \frac{X^2 Y}{6} \qquad A_c y_c = \frac{X Y^2}{6}$$

$$\therefore X = \sqrt[3]{\frac{6(A_c x_c)^2}{(A_c y_c)}}$$

and

$$Y = \left(\frac{6 A_c x_c}{X^2}\right)$$

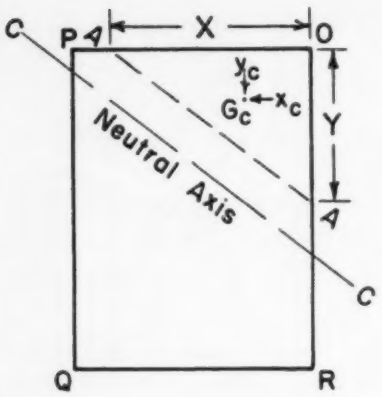

Fig. 26 — Geometry of compression zone, Case I

Case 2—Line *AA* cuts opposite sides *PQ* and *OR* as in Fig. 27.

$$A_c x_c = \frac{b^2}{2}\left[Y_1 + \frac{Y_2}{3} \right]$$

$$A_c y_c = \frac{b}{2}\left[Y_1^2 + Y_2\left(Y_1 + \frac{Y_2}{3} \right) \right]$$

$$\therefore\ Y_1 = \left[3\left(\frac{A_c x_c}{b^2} \right) - \sqrt{\frac{2}{b}(A_c y_c) - 3\left(\frac{A_c x_c}{b^2} \right)^2}\ \right]$$

and

$$Y_2 = 3\left[\frac{2}{b^2} A_c x_c - Y_1 \right]$$

Case 3—Line *AA* cuts opposite sides of *OP* and *QR* as in Fig. 28.

$$A_c x_c = \frac{t}{2}\left[X_1^2 + X_2\left(X_1 + \frac{X_2}{3} \right) \right]$$

$$A_c y_c = \frac{t^2}{2}\left[X_1 + \frac{X_2}{3} \right]$$

$$\therefore\ X_1 = \left[3\left(\frac{A_c y_c}{t^2} \right) - \sqrt{\frac{2}{t}(A_c x_c) - 3\left(\frac{A_c y_c}{t^2} \right)} \right]$$

$$X_2 = 3\left[\frac{2}{t^2} A_c y_c - X_1 \right]$$

Case 4—Line *AA* cuts adjacent sides *PQ* and *QR* as in Fig. 29. Use *Q* as origin for x, y, and e.

$$P_u e_x - F_{sc} x_{sc} + F_{st} x_{st} = 0.85 f_c' A_c x_c \quad \text{(11.2A)}$$

$$P_u e_y - F_{sc} y_{sc} + F_{st} y_{st} = 0.85 f_c' A_c y_c \quad \text{(11.3A)}$$

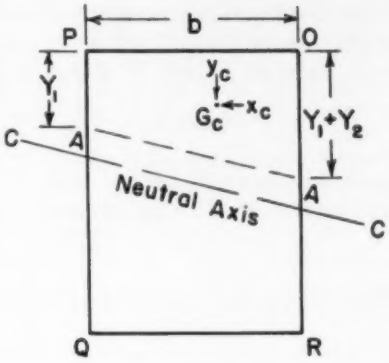

Fig. 27 — Geometry of compression zone, Case 2

$$A_c x_c = \left(\frac{b^2 t}{2} - \frac{X_1^2 Y_1}{6}\right)$$

$$A_c y_c = \left(\frac{b\, t^2}{2} - \frac{X_1 Y_1^2}{6}\right)$$

$$\therefore\ Y_1 = \frac{6}{X_1^2}\left[\frac{b^2 t}{2} - A_c x_c\right]$$

$$X_1 = \sqrt[3]{\frac{6\left[\frac{b^2 t}{2} - A_c x_c\right]^2}{\left[\frac{b\, t^2}{2} - A_c y_c\right]}}$$

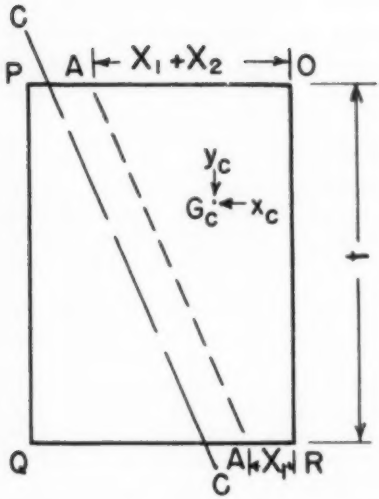

Fig. 28 — Geometry of compression zone, Case 3

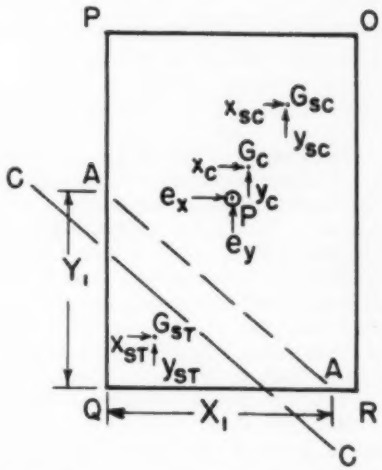

Fig. 29 — Geometry of compression zone, Case 4

Case 5—Equivalent rectangular stress distribution covers entire cross section. As in Case 4, use Q as origin. Let neutral axis CC intersect x and y axes at X' and Y' distance x' and y' from Q.

$$\text{Again } f_{sn} = E_s \varepsilon_u \left(\frac{a_n}{c}\right), \text{ which must be} \leq f_y.$$

Assume position of neutral axis. Calculate stress in each bar, and hence total steel force F_s and point of action $G_s(x_s\, y_s)$. Then:

$$P_u = F_s + 0.85 f_c' b t \quad (11.4)$$

From condition of equilibrium of internal and external moments:

$$P_u(e_x) = \{0.85 f_c' b t\} \frac{b}{2} + F_s x_s \quad (11.5)$$

and

$$P_u(e_y) = \{0.85 f_c' b t\} \frac{t}{2} + F_s y_s \quad (11.6)$$

Substituting the value of P_u from Eq. (11.4), Eq. (11.5) and (11.6) may be solved for x_s and y_s. If these values of x_s and y_s do not agree with the values calculated above using the assumed position of neutral axis, a new position of neutral axis must be assumed and the process be repeated as often as is necessary. In a simple section it may be possible to link x_s and y_s to x' and y' algebraically, in which case iteration may be used to solve the problem rather than trial and error.

Test results for columns subject to axial load and bending about both principal axes are rather scarce. The analysis proposed has been applied to ten columns tested by Andersen and Lee[27] and the results are contained in Table A-11. The average value of P_{test}/P_{calc} for the series is 0.99, and the standard deviation is 0.046.

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APPENDIX — TEST DATA

TABLE A-1—REINFORCED CONCRETE BEAMS REINFORCED IN TENSION ONLY (Ultimate strength controlled by compression)

Source	Beam No.	f_c', psi	f_y, ksi	p	$\frac{M_{ult}}{bd^2 f_c'}$ Test	$\frac{M_{ult}}{bd^2 f_c'}$ Calculated	Test / Calculated
W. A. Slater and I. Lyse[13]	1	1390	64.8	0.021	0.506	0.362	1.40
	2	2790	64.8	0.028	0.337	0.342	0.99
	3	4070	64.8	0.037	0.326	0.336	0.97
	4	4800	64.8	0.047	0.339	0.333	1.02
	5	5740	64.8	0.056	0.320	0.323	0.99
	6	2590	64.8	0.030	0.422	0.349	1.21
	6A	4130	64.8	0.039	0.327	0.337	0.97
	7	2950	64.8	0.028	0.341	0.339	1.01
	8	2760	64.8	0.031	0.380	0.348	1.09
	9	2820	64.8	0.032	0.391	0.347	1.13
	10	2820	64.8	0.030	0.346	0.345	1.00
	10A	3810	64.8	0.040	0.354	0.344	1.03
Columbia University[14]	C1	3550	61.37	0.0341	0.406	0.339	1.20
	C11	3550	62.00	0.0345	0.394	0.340	1.16
	C2	3550	63.04	0.0334	0.386	0.338	1.14
	C12	3550	64.28	0.0328	0.365	0.337	1.08

TABLE A-1 (cont.) — REINFORCED CONCRETE BEAMS REINFORCED IN TENSION ONLY (Ultimate strength controlled by compression)

Source	Beam No.	f_c', psi	f_y, ksi	p	$\frac{M_{ult}}{bd^2 f_c'}$		Test / Calculated
					Test	Calculated	
K. C. Cox[15]	122	1700	53.4	0.0176	0.405	0.343	1.18
	123	1700	53.4	0.0264	0.423	0.363	1.16
	124	1700	53.4	0.0352	0.455	0.374	1.22
	125	1700	53.4	0.0440	0.471	0.382	1.23
	142	1700	48.1	0.0244	0.442	0.359	1.23
	143	1700	48.1	0.0368	0.426	0.376	1.13
	144	1700	48.1	0.0488	0.490	0.385	1.27
	224	3100	53.4	0.0352	0.343	0.348	0.98
	225	3100	53.4	0.0440	0.353	0.359	0.98
	243	3100	48.1	0.0368	0.370	0.350	1.06
	244	3100	48.1	0.0488	0.375	0.363	1.03
	214	3100	55.2	0.0308	0.374	0.341	1.10
	215	3100	55.2	0.0388	0.372	0.353	1.05
	235	3100	48.1	0.0388	0.381	0.353	1.08
	252	3100	50.6	0.0348	0.379	0.348	1.09
	253	3100	50.6	0.0520	0.388	0.366	1.06
	325	4500	53.4	0.0440	0.320	0.336	0.95
	343	4500	48.1	0.0368	0.297	0.326	0.91
	344	4500	48.1	0.0488	0.342	0.341	1.00
	425	5800	53.4	0.0440	0.293	0.309	0.94
	444	5800	48.1	0.0488	0.300	0.314	0.95
S. L. Lash and J. W. Brison[16]	4205	1970	39.2	0.0400	0.449	0.374	1.20
	4206	1930	44.4	0.0475	0.464	0.380	1.22
	4308	3330	42.8	0.0454	0.367	0.357	1.03
	4407	4170	40.8	0.0367	0.305	0.333	0.91
	4408	4490	43.4	0.0471	0.346	0.339	1.02
	6203	2150	88.0	0.0140	0.346	0.316	1.09
	6204	2150	75.8	0.0200	0.333	0.338	0.99
	6205	1950	75.8	0.0225	0.415	0.349	1.19
	6206	2080	73.5	0.0284	0.400	0.357	1.12
	6207	1915	75.2	0.0385	0.391	0.373	1.05
	6208	2120	75.2	0.0391	0.407	0.370	1.10
	6303	3290	72.0	0.0147	0.270	0.292	0.92
	6304	2760	75.8	0.0233	0.347	0.332	1.04
	6305	3200	74.0	0.0286	0.338	0.335	1.01
	6306	2760	75.2	0.0394	0.359	0.359	1.00
	6404	4490	75.8	0.0226	0.263	0.296	0.89
	6405	4140	74.0	0.0280	0.293	0.318	0.92
	6406	4190	75.2	0.0390	0.317	0.336	0.94
	6407	4190	62.1	0.0408	0.314	0.338	0.93
	6504	4870	75.8	0.0233	0.247	0.289	0.85
	6505	4450	65.0	0.0371	0.299	0.327	0.91
	6506	5450	75.8	0.0458	0.292	0.318	0.92

Avg, all beams (59)	1.06
Standard deviation, all beams	±0.113
Avg for beams with $f_c' > 2000$ psi	1.02
Standard deviation for beams with f_c' 2000 psi	0.083

TABLE A-2—REINFORCED CONCRETE BEAMS REINFORCED IN TENSION AND COMPRESSION

Source	Beam	b	d	d'	A_s	A_s'	p, percent	p', percent	f_c', psi	f_y, ksi	f_y', ksi	$\frac{M}{bd^2f_c'}$ Test	$\frac{M}{bd^2f_c'}$ Calc	Test / Calc	Failure modes*
Gaston[17]†	c2w	6	10.58	0.90	0.88	0.40	1.38	0.63	3940	45.4	44.5	0.153	0.148	1.03	2
	c2xm	6	10.58	1.29	0.88	0.40	1.38	0.63	4070	53.3	47.0	0.175	0.163	1.07	2
(tension	c3w	6	10.37	1.48	2.00	1.20	3.22	1.93	4310	41.8	46.7	0.275	0.275	1.00	2
failures)	c3xm	6	10.37	1.48	2.00	1.20	3.22	1.93	3890	41.8	42.5	0.323	0.303	1.07	2†
	c3yna	6	10.51	1.41	1.20	0.62	1.90	0.98	3330	45.2	56.1	0.236	0.230	1.03	2
	c3ynb	6	10.37	1.55	2.00	1.20	3.22	1.93	4860	42.1	47.4	0.242	0.245	0.99	2
	c4xna	6	10.51	1.22	1.20	0.62	1.90	0.98	2450	45.5	41.4	0.310	0.313	0.99	1†
	c4xnb	6	10.51	1.51	1.20	0.88	1.90	1.39	2430	46.4	44.1	0.324	0.319	1.02	2
	c4zn	6	10.37	1.48	2.00	1.20	3.22	1.93	3570	41.3	46.4	0.329	0.326	1.01	2
	c5yn	6	9.38	1.57	3.16	1.58	5.61	2.80	4480	44.0	43.4	0.457	0.463	0.99	1†
	c6xm	6	9.38	1.57	3.16	1.58	5.61	2.80	3680	41.8	40.2	0.531	0.526	1.01	1†
	c7w	6	9.38	1.57	3.16	1.58	5.61	2.80	3480	41.6	43.6	0.552	0.552	1.00	1†
McCol-	B-34	6	10.58	1.40	0.88	0.40	1.39	0.63	4760	46.5	47.2	0.139	0.125	1.00	2
lister[18]‡	B-35	6	10.51	1.41	1.20	0.62	1.90	0.98	2345	48.4	47.2	0.347	0.343	1.01	1†
	T-1	6	10.58	1.28	0.88	0.40	1.39	0.63	3897	41.8	47.9	0.148	0.138	1.07	2
(tension	T-2	6	10.37	1.51	2.00	1.20	3.21	1.93	3858	45.4	50.0	0.334	0.330	1.01	2
failures)	T-3	6	10.20	1.56	3.12	1.58	5.10	2.58	4266	44.7	46.1	0.455	0.453	1.01	1†
	T-7	6	10.58	1.42	0.88	0.88	1.39	1.39	4540	40.9	42.5	0.127	0.116	1.09	2
	T-10	6	10.58	1.33	0.88	0.22	1.39	0.35	4330	42.6	46.3	0.142	0.126	1.13	2
	T-12	6	10.28	1.84	2.54	2.00	4.12	3.24	4367	46.0	45.8	0.390	0.369	1.06	2
	T-14	6	10.58	1.52	0.88	0.88	1.39	1.39	4030	41.4	40.9	0.145	0.131	1.00	2
	T-15	6	10.58	1.50	0.88	0.88	1.39	1.39	3700	47.0	40.5	0.171	0.159	1.07	2
	T-4	6	10.65	1.28	0.62	0.40	0.97	0.63	2230	47.1	45.7	0.190	0.186	1.02	2
	T-5	6	10.51	1.35	1.20	0.62	1.90	0.98	2021	48.4	46.6	0.407	0.396	1.03	1†
	T-6	6	10.37	1.49	2.00	1.20	3.21	1.93	1905	56.5	49.7	0.802	0.790	1.01	2
Gural-	IB-1	23	11.81	2.31	1.20	1.20	0.442	0.442	2440	84.6	84.6	0.157	0.135	1.16	2
nick[19]	IIB-1	23	11.81	2.31	1.20	1.20	0.442	0.442	2440	84.6	84.6	0.197	0.135	1.46	2
	IID-1	23	12.05	2.81	2.08	2.08	0.750	0.750	5520	87.7	87.7	0.133	0.112	1.19	2
(tension	IIIA-1	23	11.81	2.74	2.08	2.08	0.765	0.765	3130	87.7	87.7	0.216	0.188	1.15	2
failures)	IIIB-1	23	11.81	2.31	1.20	1.20	0.442	0.442	3130	84.6	84.6	0.161	0.112	1.44	2
	IIIB-2	23	12.12	2.63	1.20	1.20	0.430	0.430	3130	84.6	84.6	0.116	0.110	1.05	2
	IIIC-1	23	11.78	2.68	3.73	3.73	1.380	1.380	5300	83.9	83.9	0.203	0.207	0.98	2
	IIID-1	23	11.81	2.72	2.08	2.08	0.766	0.766	5300	87.7	87.7	0.159	0.125	1.27	2

	IIIC-1	23	11.78	2.66	3.75	3.75	1.380	1.380	5300	83.9	83.9	0.203	0.207	0.98	2
	IIID-1	23	11.81	2.72	2.08	2.08	0.766	0.766	5300	87.7	87.7	0.159	0.125	1.27	2
John-	2	4.02	5.40	0.67		0.089	4.6	0.41	2860	96.0	53.8	0.503	0.425	1.18	3
son[20]	6	3.94	5.28	0.67		0.089	4.6	0.43	2820	96.0	51.1	0.442	0.426	1.04	3
	10	4.02	5.28	0.94		0.089	4.6	0.42	3200	96.0	51.1	0.427	0.408	1.05	3
(Com-	14	3.98	5.28	0.67		0.089	4.6	0.42	2830	96.0	51.5	0.435	0.438	0.99	3
pression	3	4.14	5.12	0.71		0.244	4.6	1.15	3240	96.0	46.1	0.497	0.488	1.02	3
failures)	7	3.98	5.28	0.67		0.244	4.6	1.16	2780	96.0	46.1	0.585	0.520	1.12	3
	11	3.94	5.24	0.67		0.244	4.6	1.20	2780	96.0	39.0	0.530	0.498	1.06	3
	15	3.94	5.24	0.83		0.244	4.6	1.18	3020	96.0	39.0	0.493	0.478	1.03	3
	4	3.94	5.24	0.55		0.528	4.6	2.56	2900	96.0	41.8	0.637	0.663	0.96	3
	12	3.94	5.24	0.51		0.528	4.6	2.56	2790	96.0	43.0	0.712	0.693	1.03	3
	16	4.02	5.32	0.87		0.528	4.6	2.47	2810	96.0	42.5	0.590	0.650	0.91	3

*(1) Both tension and compression reinforcement yielded before ultimate strength; (2) Only tension reinforcement yielded; (3) Only compression reinforcement yielded.

† Test failure moment in this series is taken as maximum moment before strain hardening of tension reinforcement commenced.

‡ Test failure moment in this series is taken as the moment recorded at which the concrete in the compression zone crushed.

In both the above series a further small increase in moment was obtained in several cases due to strain hardening of the tension reinforcement. However, this increase in load was accompanied by extremely large deflections and therefore was neglected. It should be noted that strain hardening of the structural or intermediate grade steel reinforcement only occurred in cases where either a very small percentage of tension reinforcement was provided or where an excessive amount of compression reinforcement was provided such that the compression reinforcement did not yield. For normal amounts of steel used in practice, the ultimate moment was not increased by strain hardening.

TABLE A-3—REINFORCED CONCRETE T-BEAMS REINFORCED IN TENSION ONLY

Source	Beam	b, in.	d, in.	b', in.	t, in.	A_s	p, percent	f_c', psi	f_y, ksi	$\frac{M}{bd^2 f_c'}$ Test	$\frac{M}{bd^2 f_c'}$ Calc	$\frac{M_{test}}{M_{calc}}$
A. N. Talbot[21]	1	16	10	8	3.25	1.68	1.05	1890	54.9	0.305	0.250	1.22
	2	32	10	8	3.25	3.36	1.05	1870	53.8	0.269	0.248	1.08
	3	24	10	8	3.25	2.24	0.93	1760	52.7	0.262	0.233	1.12
	4	16	10	8	3.25	1.76	1.10	1330	38.3	0.296	0.258	1.15
	5	32	10	8	3.25	3.36	1.05	1190	53.4	0.435	0.340	1.28
	6	24	10	8	3.25	2.20	0.92	1610	38.3	0.200	0.193	1.03
	7	16	10	8	3.25	1.76	1.10	1450	38.3	0.249	0.244	1.02
	8	24	10	8	3.25	2.20	0.92	1750	40.7	0.187	0.187	1.00
	9	32	10	8	3.25	3.08	0.97	1610	38.3	0.195	0.197	0.99
S. A. Guralnick[19]	IA-IR	23	11.81	7	4	2.08	0.766	3230	87.7	0.200	0.182	1.10
	IB-IR	23	11.81	7	4	1.20	0.442	2440	84.6	0.184	0.139	1.32
	IC-IR	23	11.78	7	4	3.72	1.375	4930	83.9	0.205	0.202	1.02
	ID-IR	23	11.81	7	4	2.08	0.766	4930	87.7	0.138	0.125	1.10
J. R. Gaston and E. Hognestad[22]	1	9	16.25	3.5	2.75(avg)	1.20	0.82	4730	90.0	0.149	0.134	1.11
	2	9	16.00	3.5	2.75(avg)	1.60	1.11	5230	90.0	0.185	0.159	1.16

Avg $M_{test}/M_{calc} = 1.11$. Standard deviation = 0.10.

TABLE A-4—PRESTRESSED CONCRETE BEAMS WITHOUT BOND

Source	Beam No.	b, in.	d, in.	p, percent	f_{se}, ksi	Steel tensile strength, ksi	f_c', psi	$\frac{M_{ult}}{bd^2 f_c'}$ Test	$\frac{M_{ult}}{bd^2 f_c'}$ Calc	$\frac{M_{test}}{M_{calc}}$
J. R. Janney, E. Hognestad, and D. McHenry[23]	3—0.128	6.0	8.3	0.322	110	235	5900	0.084	0.079	1.06
	3—0.144	6.0	8.3	0.322	122	235	5250	0.102	0.093	1.10
	3—0.307	6.0	8.3	0.644	130	235	4930	0.200	0.184	1.09
	3—0.428	6.0	8.3	0.965	120	235	5300	0.229	0.233	0.98
	3—0.428	6.0	8.3	0.965	133	235	5300	0.238	0.241	0.99
A. Feldman[24]	U-1	5.99	8.42	0.346	127.5	250.76	4240	0.139	0.135	1.03
	U-3	6.00	7.85	0.183	120.8	250.76	5620	0.055	0.048	1.15
	U-4	6.16	8.33	0.782	118.9	250.76	4060	0.274	0.250	1.10
	U-5	6.09	8.24	0.685	120.7	250.76	4060	0.273	0.271	1.01
	U-7	6.00	7.50	0.382	121.9	250.76	5020	0.120	0.124	0.97
	U-8	6.00	7.65	0.624	118.9	250.76	2565	0.285	0.290	0.95
	U-9	6.00	7.35	0.650	120.0	250.76	3550	0.273	0.246	1.11
	U-10	6.02	7.55	0.820	118.8	250.76	3390	0.328	0.292	1.12
	U-11	5.90	8.20	0.355	122.8	250.76	5490	0.121	0.107	1.13
	U-12	6.00	8.34	0.401	125.8	250.76	4020	0.173	0.145	1.19
	U-14	5.95	7.84	0.184	122.1	250.76	7600	0.049	0.043	1.14
	U-16	5.96	7.56	0.509	121.1	250.76	2190	0.308	0.287	1.07
R. J. Allen[25]	U-17	6.20	7.39	0.527	118.8	255	2120	0.330	0.304	1.09
	U-18	6.10	7.45	0.797	110.5	255	2770	0.339	0.321	1.06
	U-19	6.04	6.85	0.365	123.7	255	6270	0.113	0.101	1.12
	U-21	6.20	7.60	0.384	119.8	255	2450	0.218	0.223	0.98
	U-22	6.10	7.32	0.743	118.2	255	4910	0.227	0.215	1.05
	U-23	6.00	6.99	0.288	117.7	255	7580	0.049	0.067	0.72
	U-24	6.00	7.85	0.192	119.0	255	5660	0.064	0.061	1.05

Avg M_{test}/M_{calc} for 24 beams = 1.05. Standard deviation = 0.094.

TABLE A-5—PRESTRESSED CONCRETE BEAMS WITH WELL BONDED TENDONS

Source	Beam No.	b, in.	d, in.	p, percent	f_{se}, ksi	Steel tensile strength, ksi	f_c', psi	$\frac{M_{ult}}{bd^2 f_c'}$ Test	$\frac{M_{ult}}{bd^2 f_c'}$ Calc	$\frac{M_{test}}{M_{calc}}$
D. F. Billet and H. H. Appleton[12]	B2	6.15	9.53	0.198	116.8	246	5420	0.087	0.079	1.10
	B3	6.00	9.62	0.101	120.0	246	3750	0.065	0.061	1.06
	B7	6.13	8.09	0.942	112.8	248	5910	0.272	0.256	1.06
	B8	6.13	7.99	0.953	112.9	248	3280	0.462	0.349	1.32
	B9	6.06	9.23	0.418	19.1	240	6330	0.129	0.128	1.01
	B10	6.06	9.01	0.107	19.0	240	3530	0.068	0.068	1.00
	B11	6.06	9.21	0.419	20.4	240	3910	0.208	0.190	1.09
	B13	6.02	8.15	0.656	21.2	240	3750	0.283	0.242	1.17
	B14	6.00	7.99	0.916	20.2	240	3750	0.327	0.266	1.23
	B15	6.03	9.29	0.418	150.0	240	5710	0.143	0.143	1.00
	B16	6.01	9.00	0.108	150.3	240	3330	0.077	0.073	1.05
	B17	6.00	9.09	0.429	151.0	240	4580	0.179	0.178	1.01
	B18	6.00	8.29	0.647	148.8	240	4100	0.273	0.264	1.03
	B19	6.08	8.27	0.873	151.3	240	6220	0.244	0.242	1.01
	B21	6.08	9.05	0.284	118.0	248	6560	0.093	0.098	0.95
	B22	6.07	9.13	0.561	115.2	248	7630	0.153	0.150	1.02
	B23	6.04	8.20	0.943	117.3	248	8200	0.213	0.206	1.05
	B24	6.07	8.24	0.746	116.4	248	6120	0.235	0.221	1.06
	B25	6.06	8.01	0.641	114.5	248	3270	0.349	0.294	1.19
	B27	6.07	8.36	0.920	118.0	248	4590	0.319	0.295	1.08
A. Feldman[24]	B28	6.15	7.93	0.475	92.5	186	2500	0.281	0.260	1.08
	B29	6.16	8.07	0.815	92.7	186	4280	0.261	0.240	1.09
	B30	6.09	8.08	0.177	101.1	248	2890	0.135	0.129	1.05
	B31	6.08	8.23	0.579	94.1	248	3450	0.279	0.250	1.12
	B32	6.00	9.32	0.510	115.3	256	7180	0.154	0.158	0.97
	B33	6.03	9.08	0.312	116.9	256	8320	0.093	0.091	1.02
J. R. Janney, E. Hognestad, and D. McHenry[23]	1—0.141	6	8.3	0.322	119	235	5350	0.144	0.127	1.13
	1—0.250	6	8.3	0.644	113	235	6050	0.218	0.186	1.17
	1—0.420	6	8.3	0.965	117	235	5400	0.260	0.258	1.01
	2—0.151	6	8.3	0.322	126	235	5000	0.146	0.134	1.07
	2—0.306	6	8.3	0.644	118	235	4950	0.237	0.214	1.11
	2—0.398	6	8.3	0.965	117	235	5700	0.271	0.251	1.08

Average M_{test}/M_{calc}: All 32 beams = 1.07. Standard deviation = 0.077.

TABLE A-6—UNSYMMETRIC REINFORCED CONCRETE T-BEAMS

Source	Beam	Flange width, B, in.	Effective depth, d, in.	f_c', psi	A_s, sq in.	f_y, ksi	Ultimate moment, in.-kip		$\frac{M_{test}}{M_{calc}}$	Type of failure
							Calc	Test		
A. H. Mattock and L. B. Kriz[28]	L1	8	13.8	2820	1.20	45.6	647	678	1.05	T
	L2	8	13.8	6200	1.20	46.1	710	693	0.98	T
	L3	8	12.9	3260	2.40	45.6	952	1062	1.12	C
	L4	8	12.9	5570	2.40	51.0	1300	1325	1.02	CT
	L5	12	13.8	2050	1.20	47.9	627	671	1.07	T
	L6	12	13.8	6320	1.20	47.5	733	778	1.06	T
	L7	12	12.9	2225	2.40	43.0	695	876	1.26	C
	L8	12	12.9	6420	2.40	43.0	1160	1173	1.01	T
	L9	16	13.8	2680	1.20	45.4	637	663	1.04	T
	L10	16	13.8	4820	1.20	44.5	678	757	1.12	T
	L11	16	12.9	2530	2.40	50.7	775	1129	1.46	C
	L12	16	12.9	5820	2.40	50.5	1298	1265	0.97	T

(a) All beams: Avg $M_{test}/M_{calc} = 1.10$; Standard deviation $= 0.138$.
(b) Beams with ultimate strength controlled by tension: Avg $M_{test}/M_{calc} = 1.04$; Standard deviation $= 0.047$.

TABLE A-7—REINFORCED CONCRETE BEAMS WITH TRIANGULAR COMPRESSION ZONES

Source	Beam	f_y, psi	f_y', ksi	A_s, sq in.	Ultimate moment, in.-kip		$\frac{M_{test}}{M_{calc}}$
					Calc	Test	
A. H. Mattock and L. B. Kriz[28]	T1	3620	49.0	0.88	194.5	196.8	1.01
	T2	3455	49.0	1.32	215.0	240.9	1.12
	T3	6290	45.0	1.80	320.0	326.8	1.02
	T4	1735	49.5	0.88	116.3	131.8	1.13
	T5	3500	50.5	1.32	219.0	258.4	1.18
	T6	7000	45.0	1.80	323.0	368.3	1.14

For all beams, $d = 7$ in., $\theta = 45$ deg.
Ultimate strength of all beams controlled by crushing of concrete.
Avg $M_{test}/M_{calc} = 1.10$. Standard deviation $= 0.069$.

TABLE A-8—TIED REINFORCED CONCRETE COLUMNS LOADED CONCENTRICALLY*

Column	Percentage reinforcement	Yield point of reinforcement, psi	f_c', psi	Ultimate load, kip		$\frac{P_{test}}{P_{calc}}$
				Test	Calc	
a	4	50,000	2860	219	231	0.95
b	4	50,000	3090	255	242	1.05
a	4	50,000	2650	253	222	1.14
b	4	50,000	2850	238	231	1.03
a	1.5	44,700	4700	225	246	0.92
b	1.5	44,700	4150	227	222	1.02
a	4	50,000	4670	285	310	0.92
b	4	50,000	4730	320	313	1.02
a	4	50,000	4225	293	291	1.01
b	4	50,000	4570	309	306	1.01

Continued on p. 926

TABLE A-8 (cont.)—TIED REINFORCED CONCRETE COLUMNS LOADED ECCENTRICALLY*

Column	Percentage reinforcement	Yield point of reinforcement, psi	f_c', psi	Ultimate load, kip		$\frac{P_{test}}{P_{calc}}$
				Test	Calc	
a	6	42,200	4215	317	315	1.01
b	6	42,200	4985	291	348	0.84
a	4	50,000	5870	353	363	0.97
b	4	50,000	6950	387	410	0.94
a	4	50,000	6245	410	379	1.08
b	4	50,000	6530	420	391	1.08

Average value of $P_{test}/P_{calc} = 1.00$. Standard deviation = 0.074.
*Source: Series 3 of ACI column investigation—University of Illinois *Bulletin* No. 267.

TABLE A-9—RECTANGULAR REINFORCED CONCRETE COLUMNS LOADED ECCENTRICALLY

Group I

Source and characteristics	Column No.	Concrete cylinder strength, f_c', psi	Eccentricity (total), in.	Ultimate load, kip		$\frac{P_{test}}{P_{calc}}$	Mode of failure
				Test	Calc		
E. Hognestad[8]	A-1a	5280	0.12	388	452	0.86	C
	A-1b	5660	0.14	441	481	0.92	C
$b = t = 10$ in.	B-1a	4250	0.12	343	372	0.92	C
	B-1b	4070	0.12	352	356	0.99	C
$A_s = 1.24$ sq in.	C-1a	2270	0.13	222	212	1.05	C
	C-1b	2020	0.13	191	192	1.00	C
$A_s' = 0.22$ sq in.	A-2a	5280	2.72	239	240	1.00	C
	A-2b	5830	2.78	253	260	0.97	C
$f_y = 43.6$ ksi	B-2a	4250	2.77	213	206	1.03	C
	B-2b	4070	2.74	190	197	0.96	C
$E_s = 28 \times 10^6$ psi	C-2a	2270	2.77	118.5	116	1.02	C
	C-2b	1970	2.77	100.0	103	0.97	C
$f_y' = 60.0$ ksi	A-3a	5660	5.32	133.5	154.0	0.87	T
	A-3b	5830	5.28	140.0	158.0	0.89	T
$d = 8.67$ in.	B-3a	4630	5.41	125.9	134.3	0.94	CT
	B-3b	4290	5.37	116.0	129.4	0.90	CT
$d' = 1.33$ in.	C-3a	1880	5.28	60.5	66	0.92	C
	C-3b	1690	5.33	64.0	52	1.03	C
	A-4a	4810	7.95	84.5	85.7	0.99	T
	A-4b	5600	7.85	81.0	92.1	0.88	T
	B-4a	3800	7.98	80.0	77.7	1.03	T
	B-4b	4290	8.02	81.0	81.1	1.00	T
	C-4a	1690	7.82	50.5	48	1.05	C
	C-4b	1730	7.81	52.0	49	1.06	C
	A-5a	4810	12.90	48.2	45.6	1.06	T
	A-5b	5600	12.90	42.8	46.9	0.91	T
	B-5a	4290	12.92	46.1	44.5	1.04	T
	B-5b	4590	12.95	45.5	45.0	1.01	T
	C-5a	2310	12.84	39.0	37.7	1.04	CT
	C-5b	1770	12.84	32.8	34.0	0.96	CT

Group II

E. Hognestad[8]	B-6a	4080	0.07	456	437	1.04	C
	B-6b	4040	0.06	420	436	0.96	C
$b = t = 10$ in.	C-6a	2020	0.10	225	268	0.84	C
	C-6b	1520	0.18	202	222	0.91	C

TABLE A-9 (cont.)—RECTANGULAR REINFORCED CONCRETE COLUMNS LOADED ECCENTRICALLY

Source and characteristics	Column No.	Concrete cylinder strength, f_c', psi	Eccentricity (total), in.	Ultimate load, kip		$\frac{P_{test}}{P_{calc}}$	Mode of failure
				Test	Calc		
$A_s = 1.24$ sq in.	A-7a	5240	3.44	274	254	1.08	C
	A-7b	5810	2.76	284	312	0.91	C
$A_s' = 1.24$ sq in.	B-7a	4080	2.75	256	245	1.04	C
	B-7b	4040	2.74	248	244	1.02	C
$f_y = 43.6$ psi	C-7a	1970	2.78	141	147	0.96	C
	C-7b	1520	2.77	126.8	135	0.94	C
$E_s = 28 \times 10^6$ psi	A-8a	5520	5.34	162	177.9	0.91	T
	A-8b	5810	5.40	152	179.5	0.85	T
$f_y = 43.6$ psi	B-8a	4700	5.35	156	164.9	0.95	CT
	B-8b	4260	5.32	146	158.6	0.92	CT
$d = 8.67$ in.	C-8a	1820	5.32	99	103	0.96	C
	C-8b	1820	5.39	99	102	0.97	C
$d' = 1.33$ in.	A-9a	5100	7.87	89.0	99.1	0.90	T
	A-9b	5170	7.89	91.2	99.1	0.92	T
	B-9a	4700	7.85	94.0	97.7	0.96	T
	B-9b	4370	7.82	89.5	96.6	0.93	T
	C-9a	1880	7.88	73.0	75.8	0.96	CT
	C-9b	1730	7.85	65.5	74.1	0.88	CT
	A-10a	5100	12.78	46.1	47.8	0.97	T
	A-10b	5170	12.75	44.0	48.0	0.92	T
	B-10a	4260	12.78	43.5	47.2	0.92	T
	B-10b	4370	12.79	44.0	47.2	0.93	T
	C-10a	2300	12.85	44.5	44.3	1.00	T
	C-10b	1770	12.88	45.0	42.6	1.06	T
Group III							
E. Hognestad[9]	B-11a	3870	0.08	500	513	0.98	C
	B-11b	4070	0.10	485	517	0.94	C
$b = t = 10$ in.	C-11b	2070	0.00	353	376	0.94	C
	A-12a	4150	2.70	315	306	1.03	C
$A_s = 2.40$ sq in.	A-12b	5050	2.72	326	343	0.95	C
	B-12a	4300	2.72	303	318	0.95	C
$A_s' = 2.40$ sq in.	B-12b	4010	2.76	284	298	0.95	C
	C-12a	2300	2.76	252	224	1.12	C
$f_y = 43.6$ ksi	C-12b	2200	2.72	230	218	1.05	C
	A-13a	5350	5.36	220	227	0.97	C
$E_s = 29 \times 10^6$ psi	A-13b	4850	5.34	210	216	0.97	C
	B-13a	3580	5.35	180	188	0.96	C
$f_y = 43.6$ ksi	B-13b	4290	5.34	206	206	1.00	C
	C-13a	2300	5.33	151	153	0.99	C
$d = 8.50$ in.	C-13b	2070	5.28	137	148	0.93	C
	A-14a	5350	7.87	142	159.0	0.89	T
$d' = 1.50$ in.	A-14b	5100	7.93	153	155.5	0.98	T
	B-14a	3580	7.89	138.8	141.0	0.98	CT
	C-14a	1950	7.84	115.5	108	1.07	C
	C-14b	2070	7.87	104.0	111	0.94	C
	A-15a	5100	12.92	88.0	82.8	1.06	T
	A-15b	4850	12.85	79.0	83.0	0.95	T
	B-15a	3800	12.91	74.0	80.3	0.92	T
	B-15b	4630	12.92	84.5	82.0	1.03	T
	C-15a	1950	12.89	72.5	74	0.97	C
	C-15b	2070	12.91	74.5	73.5	1.01	CT

Avg $P_{test}/P_{calc} = 0.97$. Standard deviation = 0.059.

Note: The eccentricity recorded in this table is measured from the center of the column.

TABLE A-10—CIRCULAR SECTION COLUMNS SUBJECT TO ECCENTRIC LOAD*

Column No.	f_c', psi	Eccentricity, e, in.	Ultimate load, kips		$\frac{P_{test}}{P_{calc}}$	Cross section
			Test	Calc		
A-16a	5150	0	760	704	1.08	All columns 12-in. diameter.
A-16b	4640	0.01	693	655	1.06	
B-16a	2990	0.03	515	497	1.03	Reinforcement: eight ⅞-in. diameter
B-16b	3310	0.01	514	527	0.98	
C-16a	1590	0.03	371	362	1.02	
C-16b	1420	0.02	365	345	1.06	f_y = 43.8 ksi
A-17a	5150	3.30	343	308.6	1.11	
A-17b	4640	3.29	283	291.9	0.97	
B-17a	3620	3.34	253	251.7	1.01	
B-17b	3310	3.34	238	239.3	0.99	
C-17a	1420	3.55	187	160.0	1.17	
C-17b	1600	3.50	179	164.9	1.08	
A-18a	5020	6.44	162	164.3	0.99	
A-18b	5000	6.50	171	162.5	1.05	
B-18a	3380	6.42	140	140.8	0.99	
B-18b	3580	6.47	136	143	0.95	
C-18a	1680	6.80	127	100.6	1.26	
C-18b	1590	6.60	107	101.9	1.05	
A-19a	5020	9.62	111.0	107.2	1.04	
A-19b	5310	9.62	114.3	109.4	1.04	
B-19a	3380	9.54	98.5	93.2	1.06	
B-19b	3580	9.56	103.0	95.4	1.08	
C-19a	1680	9.80	79.0	72.9	1.08	
C-19b	1630	9.80	79.0	72.4	1.09	
A-20a	5310	15.68	67.7	63.0	1.07	
A-20b	5000	15.58	63.5	62.0	1.02	
B-20a	2990	15.75	57.5	53.0	1.08	
B-20b	3620	15.60	62.0	57.0	1.09	
C-20a	1630	15.60	47.0	45.8	1.03	
C-20b	1600	15.72	47.0	44.9	1.05	

Avg P_{test}/P_{calc} for all 30 columns = 1.05
Standard deviation = 0.060
*Source: E. Hognestad.[5]

TABLE A-11 — RECTANGULAR COLUMNS SUBJECT TO AXIAL THRUST AND BENDING ABOUT TWO AXES*

Column No.	b, in.	r, in.	d', in.	Reinforcement	f_y, psi	f_c', psi	e_x', in.	e_y', in.	P_{test}, kip	P_{calc}, kip	$\frac{P_{test}}{P_{calc}}$
SC1	4	4	0.75	4–1/4 diameter	35.8	5435	3.14	3.14	5.27	5.10	1.03
SC6									4.68		0.92
SC2	4	4	0.75	4–5/16 diameter	39.2	5435	3.07	3.07	7.44	7.46	1.00
SC7									7.25		0.97
SC3	4	4	0.75	4–3/8 diameter	40.2	5435	3.06	3.05	9.50	9.13	1.04
SC8									9.81		1.07
SC4	4	4	0.75	4–1/2 diameter	45.7	5435	3.03	3.03	13.50	13.88	0.97
SC9									14.30		1.03
SC5	4	4	0.75	4–5/8 diameter	40.7	5435	3.02	3.02	16.48	17.34	0.95
SC10									16.50		0.95

Avg P_{test}/P_{calc} = 0.99 Standard deviation = 0.046
*Source: P. Anderson and H. N. Lee.[27]

Title No. 57-44

Creep of Prestressed Concrete Beams

By W. S. COTTINGHAM, P. G. FLUCK, and G. W. WASHA

Presents test results obtained during 7 years of sustained loading of six prestressed concrete beams. Each beam was end-supported over a 20-ft span and uniformly loaded. Two beams were loaded to full design load, two were loaded to ¾ design load, and two were loaded to ½ design load. Creep deflections and creep strains measured during the 7 year loading period are reported. Deflection and strain recoveries due to removal of the loads and for 64 days after load removal as well as the final failure loads for the beams are reported.

■ A PRECAST, PRESTRESSED CONCRETE BEAM was developed for and manufactured by the F. Hurlbut Co., Green Bay, Wis., for general use in small structures. The results of "short-time" bending tests were used to arrive at the final section and design load values. A cross section of the beam is shown in Fig. 1. Because of the lack of data on the creep characteristics of prestressed concrete, six beams were subjected to sustained load for 7 years. The six beams were manufactured at the F. Hurlbut Co. plant with the same materials and at the same time to eliminate manufacturing variables as far as possible.

MATERIALS

Concrete

The concrete, designed to have an ultimate compressive strength of about 6000 psi at 28 days, had weight proportions of 1 part high-early-strength portland cement to 2¼ parts of sand to 2⅓ parts of stone with a maximum size of ¾ in. The water-cement ratio of the fresh concrete was about 0.50 by weight. After placement the concrete was subjected to a vacuum process.

Steel

Each of the eight steel reinforcing cables was made of seven strands of No. 14 wire. Each cable had an effective cross-sectional area of 0.0352 sq in. The individual wires had ultimate tensile strengths of 287,500 psi. The average ultimate tensile strength of the cables was 278,000 psi and the average modulus of elasticity was 25,600,000 psi. In addition to the reinforcing cables each beam had two #2 bars of

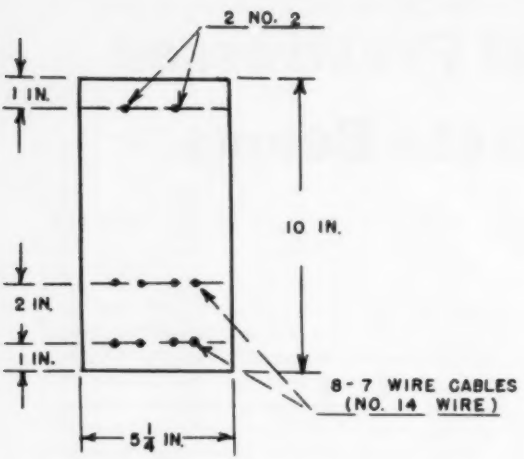

Fig. 1 — Cross section of beam

intermediate grade steel set 1 in. below the top surface to take care of handling stresses. Shear reinforcement consisted of 3 x 8—2/5 welded wire fabric along each side of the beam.

MANUFACTURE

The 5¼ x 10-in. beams were cast on stressing beds. Each bed had a solidly fixed anchor at one end and a motor-driven stressing apparatus at the other. The eight steel cables were positioned in the lower part of the beam, arranged in two horizontal layers with four cables in each layer. After the cables were positioned by spacer blocks the desired initial tension of 140,000 psi was applied. The forms for the concrete were then bolted into place and the handling and shear steel positioned. The concrete was placed and a vacuum was applied for 20 min at the end of which time the forms were removed. The beams were then covered with a portable canopy and kept at a temperature of about 70 F by use of steam during the first 60 hr after manufacture. At the end of this time the concrete had gained a strength of about 3500 psi and it was possible to cut the cable between beams. Thus, after 60 hr the stress in the cables was transferred by bond to the concrete.

SHORT TIME BENDING TESTS

To determine the short time load characteristics of the beams they were simply supported over a span of 20 ft and were loaded at the two quarter points. Nine beams, made at different times, were tested in this manner. Seven failed at loads between 13,000 and 15,000 lb. One of the remaining two failed at a lower load and the other at a higher load. On the basis of these tests a load of 5700 lb superimposed

TABLE 1—DEFLECTION OF BEAMS UNDER A 4000-LB LOAD

Superimposed load	Beam No.	Deflection, in.	Beam No.	Deflection, in.	Avg deflection, in.
Full Design (5700 lb)	25	0.39	29	0.47	0.43
¾ Design	23	0.46	27	0.39	0.425
½ Design	24	0.44	28	0.43	0.435

on the beam was taken as the design load. This was in addition to the weight of the beam (1100 lb).

Each beam intended for the creep test was end supported and subjected to a superimposed load of 4000 lb applied at the quarter points. The center deflections obtained with the 4000 lb loads were determined. The beams for the creep tests were paired as shown in Table 1 to reduce the effect of beam variations.

CREEP TESTS

Test procedure

The beams were placed on end supports at an age of about 2 months. Each end support consisted of a section of ¾-in. steel pipe resting on a notched steel plate which in turn was mortar-bedded on concrete blocks. A thin crushing strip of soft wood was used between the beam and the supporting pipe.

Brass plugs for reading deflections were placed in the top of the beam over each support and at the center. Elevation readings were taken to the nearest 0.01 in. with a Kern engineers' level. Plugs for reading tensile and compressive strains over a gage length of 20 in. were placed on both sides of each beam 1 in. above the bottom and

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P. G. Fluck has been on the staff of the Mechanics Department, University of Wisconsin, since 1941 except for service in the U. S. Navy and short periods with the Wisconsin Highway Commission Materials Testing Laboratory and the Timber Mechanics Division, U. S. Forest Products Laboratory. An ACI member, Professor Fluck is a member of Committee 209, Creep and Volume Changes in Concrete.

G. W. Washa, professor of mechanics, University of Wisconsin, received the Wason Medal for noteworthy research for work reported in his 1940 paper—"Comparison of the Physical and Mechanical Properties of Hand-Rodded and Vibrated Concrete Made With Different Cements." He was a member of the ACI Board of Direction, 1948-50 and 1954-56. He was chairman of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, and secretary and chairman of Committee 115, Research. He is now a member of Committee 609, Consolidation of Concrete, and Committee 613, Recommended Practice for Proportioning Concrete Mixes, and continues as a member of Committees 115 and 213.

TABLE 2—DEFLECTIONS

Superimposed load	*A* Immediate elastic deflection, in.	*B* Seven year creep deflection, in.	*B/A*
Full design	0.67	1.16	1.73
¾ design	0.47	0.68	1.45
½ design	0.31	0.38	1.23

1 in. below the top surface, respectively. Strain readings were taken with a 20-in. Berry strain gage. All strain values are based on readings on the brass plugs. No direct readings on the steel were made.

The beams were uniformly loaded by their own weight and by additional solid concrete weights hung from the beams. Two beams were uniformly loaded to ½ design load, two to ¾ design load, and two to full design load (total superimposed load of 5700 lb or 285 lb per lineal ft).

Deflection and strain readings were taken before and as soon as possible after placement of the superimposed load. Placement of the superimposed load required several hours, but the deflections and strains that occurred during the loading period have been called "immediate elastic" in this paper. Additional readings were taken at approximately 3, 7, 28, 90, 180, 270, 365 days, and then infrequently up to 7 years. During the sustained loading period the temperature in the storage room varied from about 65 to 80 F, and the relative humidity varied from about 20 to 90 percent. At the conclusion of the creep tests each beam was unloaded and recovery strain and deflection readings were taken immediately after unloading and at various times after unloading. Removal of the load from a beam required about an hour but the recoveries are referred to as "immediate." The last recovery reading was taken 64 days after unloading. Each beam was then placed in a testing machine, end supported and loaded at the quarter points, and tested to failure.

Results

The deflections due to the sustained load are shown in Fig. 2 for beams loaded to ½, ¾, and full design load for a 7 year loading period. The curves show that about 75 percent of the 7 year creep deflection was obtained in the first year. The data in Table 2 show that the ratio of 7 year creep deflection to the immediate elastic deflection varied from 1.23 to 1.73 as the loading increased from ½ design load to full design load.

Although there is no basis for a good correlation, it may be of interest to compare the creep deflection of the prestressed beams under full design load with the creep deflection of 8 x 12-in. ordinary reinforced

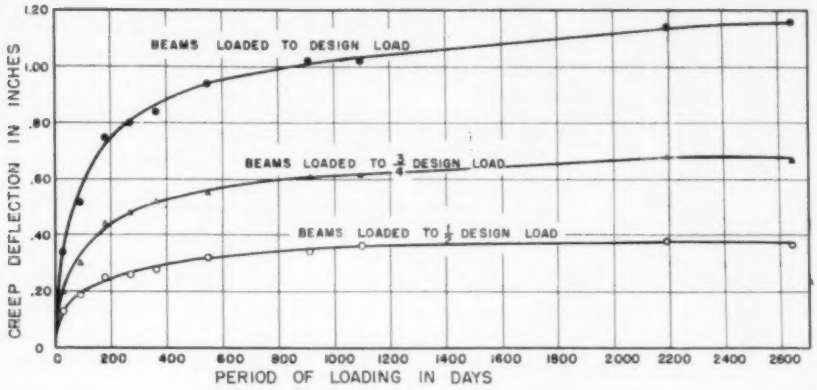

Fig. 2—Creep-deflection curves

concrete beams. The details of the two cases are shown for comparison purposes in Table 3. It may be noted that the creep deflections for the two beams are nearly equal.

Fig. 3 shows the variation in creep strains measured 1 in. above the bottom and 1 in. below the top of the beam for the seven year loading period. Each plotted value is the average of the readings on both sides of each of two companion beams. The compressive creep strains measured near the top increased at a decreasing rate with the duration of loading, and were still increasing after 7 years of sustained loading. The compressive creep strains for any given loading period were approximately proportional to the magnitude of the applied load. The creep strains measured near the bottom for the beams loaded to full design load showed increasing tensile creep strain as the loading period in-

TABLE 3—DETAILS OF PRESTRESSED AND ORDINARY CONCRETE BEAMS UNDER SUSTAINED LOAD

Details	Prestressed concrete beams	Ordinary concrete beams*
Width, in.	5¼	8
Depth, in.	10	12
Span, ft	20	20
Area of tensile steel, sq in.	0.282	1.32
28-day concrete compressive strength, psi	6000	3900
Lineal load including weight of beam, lb per ft	340	378
2½ year creep deflection, in.	1.02	1.09

*Washa, G. W., and Fluck, P. G., "The Effect of Compressive Reinforcement on the Plastic Flow of Reinforced Concrete Beams," ACI JOURNAL, V. 24, No. 2, Oct. 1952 (*Proceedings* V. 49), pp. 89-108.

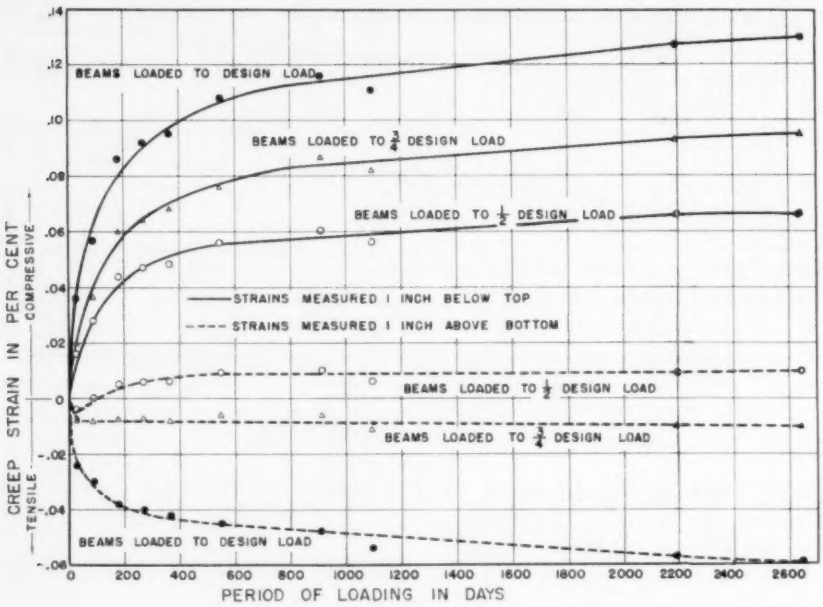

Fig. 3—Creep-strain curves

creased. At the end of the 7 year loading period these strains were still increasing. During the loading period the steel stress increase based on the measured creep strain and the modulus of elasticity of the cable was about 15,000 psi. The beams loaded to ¾ design load quickly reached a tensile creep strain near 0.01 percent and showed little further change with time. The beams loaded to ½ design load showed a slight initial tensile creep strain, but after about a 1 year loading period a compressive creep strain near 0.009 percent was reached and remained essentially constant to the end of the 7 year loading period.

Table 4 compares immediate elastic and 7 year creep strains. The ratios of the 7 year creep strains to the immediate elastic strains in compression were fairly consistent, decreasing from 3.19 for the beams loaded to ½ design load to 2.89 for those loaded to full design load. The ratios in tension were quite variable. The beams loaded to ½ design load had immediate elastic strains in tension and the 7 year creep strains were compressive. The immediate elastic and 7 year creep strains of all the other beams were tensile strains.

The recoveries in strain and deflection immediately after the load had been removed and the creep recovery during the 64 days after load removal are presented in Table 5. For comparison purposes values are also given for initial loading and for 7 years of sustained loading. The last line gives information on the residual values at the end of test.

TABLE 4—BEAM STRAINS

Super-imposed load	Measured 1 in. below top*			Measured 1 in. above bottom*		
	A Immediate elastic strain, percent	*B* Seven year creep strain, percent	*B/A*	*C* Immediate elastic strain, percent	*D* Seven year creep strain, percent	*D/C*
Full design	+0.045	+0.130	2.89	−0.045	−0.058	1.29
¾ design	+0.032	+0.095	2.97	−0.031	−0.010	0.32
½ design	+0.021	+0.067	3.19	−0.018	+0.009	—

*Tension −, Compression +.

Comparison of the values obtained immediately after unloading with those obtained immediately after initial loading shows that good agreement was obtained for the beams loaded to ½ and ¾ design load. The recovery values for the beams loaded to full design load were significantly higher than the initial values. These results suggested that the beams subjected to full design load had developed tensile cracks during the creep tests and thus were less stiff than they were when originally loaded. Such cracking must have progressed gradually over a period of time because there are no abrupt changes in the deflection and strain curves. During the final tests to determine failure loads all beams were carefully examined for cracks. Visible cracks in the beams originally loaded to full design load in the creep test were observed between loads of 400 and 1000 lb. Visible cracks in all other beams were observed between test loads of 6000 and 7000 lb, well above the loads sustained in the creep tests.

TABLE 5—DEFLECTIONS AND STRAINS DUE TO LOADING, CREEP AND UNLOADING

	Full design load			¾ design load			½ design load		
	Deflection, in.*	Strain, percent,† Top	Strain, percent,† Bottom	Deflection, in.*	Strain, percent,† Top	Strain, percent,† Bottom	Deflection, in.*	Strain, percent,† Top	Strain, percent,† Bottom
Immediate elastic loading	+0.67	+0.045	−0.045	+0.47	+0.032	−0.031	+0.31	+0.021	−0.018
Creep—7 Year	+1.16	+0.130	−0.058	+0.68	+0.095	−0.010	+0.38	+0.067	+0.009
Immediate elastic unloading	−0.84	−0.066	+0.081	−0.46	−0.035	+0.032	−0.32	−0.022	+0.019
Creep recovery—64-day	−0.08	−0.006	+0.005	−0.04	−0.004	+0.004	−0.02	−0.004	+0.002
Residual (Sum of four values above)	+0.91	+0.103	−0.017	+0.65	+0.088	−0.005	+0.35	+0.062	+0.012

*Down +, Up −.
†Tension −, Compression +.

The total recovery values (immediate elastic due to unloading plus 64-day creep recovery) were roughly 10 percent greater than the immediate recovery values. Most of the recovery took place in the first day after unloading. The residual deflections and strains were greatest for the beams subjected to sustained full design load and usually least for those subjected to ½ design load.

In the final bending tests the two beams subjected to sustained full design loads in the creep tests failed at loads of 13,800 and 12,740 lb, the two beams subjected to sustained ¾ design loads failed at loads of 12,600 and 12,600 lb, and the two beams subjected to sustained ½ design loads failed at loads of 12,100 and 13,540 lb.

CONCLUSIONS

1. The ratio of the 7 year creep deflection to the immediate elastic deflection varied between 1.23 and 1.73.
2. The creep reflections and the compressive creep strains near the top of the beam increased at a decreasing rate as the period of sustained loading increased.
3. About 75 percent of the 7 year creep deflection was obtained in the first year of loading.
4. The beams which sustained full design load developed tension cracks and changed their elastic properties.
5. Beams subjected to sustained loads that did not produce cracking during the 7 year creep tests showed essentially the same elastic behavior on unloading that they had during original loading.
6. The loads required to cause failure of the beams which had been loaded to ½, ¾, and full design load for 7 years were not significantly different.

ACKNOWLEDGMENTS

This research program was financed by funds from the University of Wisconsin Engineering Experiment Station and by additional research funds of the Mechanics Department of the University of Wisconsin. All beams were manufactured by F. Hurlbut Co., Green Bay, Wis., and were donated to the University of Wisconsin. Thanks are due to F. J. Lenfestey and his associates at the F. Hurlbut Co. for their helpful cooperation. Gerald R. Brunstrom, John V. Davis, and Thomas J. McGuan, students at the university, made some of the short-time bending tests.

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Title No. 57-45

Prestressed Precast Arches for Industrial Roof

By E. R. CANCIO and A. HERRERA

Describes the structural design of an industrial building in Havana, Cuba, to meet special requirements. Several solutions were considered but the one selected consisted of prestressed joists and tied arches having spans up to 102 ft 6 in. Prefabrication was used to a large extent. Unit costs are also given.

■ PLASTICOS INDUSTRIALES EXTRUSOS, S. A., manufacturer of plastic products, principally extruded, required a building in the vicinity of Havana to house its factory and offices. The site selected by the owners was close to a residential section, so that in addition to the general functional and economical requirements, the building had to be aesthetically in keeping with the neighborhood.

DESIGN REQUIREMENTS

Two zones entirely different in character but intimately related were required, one for the laboratory and offices and the other for manufacturing. This was accomplished by a two-story building at the front of the site, using the main floor and part of the second story for the office and the rest of the second story for the laboratory, and another building for the industrial zone (Fig. 1). The latter has three well-defined areas; the raw materials warehouse, the factory proper, and the warehouse for the finished products.

Because most of the manufactured products are extruded, it was considered convenient to have large spans in one direction, a requirement equally suitable for the warehouses as it facilitates materials handling and makes maximum use of available space. Since the manufacturing process gives off heat, a clear height of 20 ft was needed in addition to abundant cross ventilation. This clear height also makes meeting the requirements for future development of the industry possible. The general layout was studied exhaustively with a view to need for probable future expansion in a horizontal plane.

SOLUTION ADOPTED

Prestressed and reinforced concrete construction was adopted in general, but with the intention to prefabricate most of the elements since this offered the greatest economy in comparison with other methods and materials.

The office needed as much space as possible without columns to permit maximum flexibility for locating interior partitions and their possible relocation in the future. A simply supported 30 ft 6 in. span was chosen for this area, spacing the outside columns to form 20 ft bays with longitudinal beams. These beams support the prestressed joists and the cast-in-place slab. The same type of joists were used at both levels, spacing them 5 ft center to center at the first level and 6 ft 8 in. center to center for the second level in accordance with different live loads.

In the industrial area it was decided to space the columns longitudinally for total distances of 65 ft 6 in., 102 ft 6 in., 65 ft 6 in. with a depth of five bays at 20 ft, and a clear height of 20 ft.

It was decided to use prefabricated two-hinged tied arches, combined with prefabricated I-section joists to support the cast-in-place slab. This proved to be the most economical solution since it offers the following advantages: total elimination of falsework; cost of forms was reduced due to the large number of equal members; cost of placing them was reduced due to the efficiency developed by repetition; construction time was reduced by precasting the superstructure while excavation and substructure work was done; and minimum area of formwork was required for the slab due to repetition, light weight of the slab, and possibility of stripping quickly. This solution required

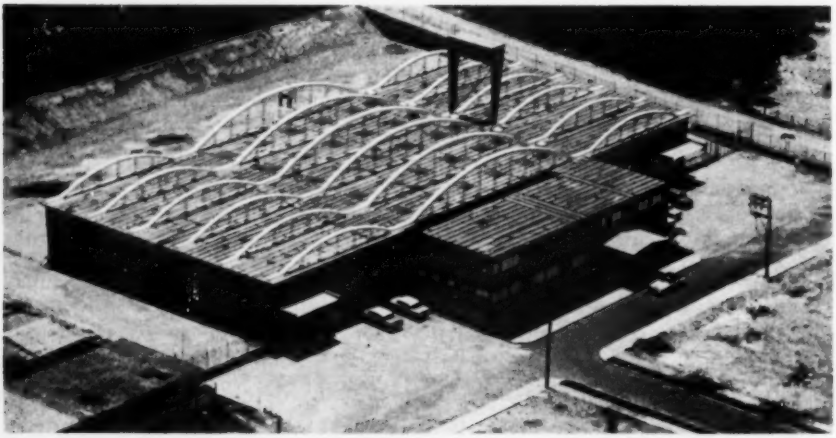

Fig. 1—General view of finished building. Note precast concrete water tank and supporting structure atop building

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the least amount of materials. At the same time the elements used were not as elaborate nor did they require as much formwork, falsework, or careful placing as would have been required in the case of continuous shells. These advantages were good not only for the long span but also for the smaller spans of 65 ft.

Prefabricated double T and channel sections were considered for the slab itself but the cast-in-place slab was adopted because the cost is approximately the same and it provides the inherent advantages of a continuous monolithic slab. The standard reinforced concrete slab is 3 in. thick. The prestressed concrete joists are spaced so as to leave a clear distance of 8 ft 3/8 in. between them; so as to be able to use 8-ft standard plywood panels, thus reducing the cost of formwork by eliminating wastage and increasing the life of the forms.

The horizontal arch tie has an inverted T-section which offers a good bearing surface for the support of the joists and sufficient vertical rigidity, without affecting the clear height of the floor below. The horizontal tie is suspended from the arch with steel bars covered with noncorrosive paint and placed inside plastic tubes. The hangers are placed on either side of the joists, avoiding undesirable flexure in the tie.

In designing the arches one of the most important considerations was the elastic stability of the upper chord, which was extremely slender. For example, if the central arch with its 20 x 16-in. section is considered as a straight column it can be observed that it has a slenderness ratio of 79.

The buckling effect was analyzed first along the directrix of the arch and then in the vertical plane which contains the directrix. The greatest slenderness occurred along the directrix of the arches in which case the buckling resistance was analyzed by Mayer's method* which gave a length equivalent to 0.532 of the real span of the arch. With this figure the slenderness ratio was reduced to 40.2, which was still greater than permitted by the 1956 ACI Code, in which Section 1107 limits the slenderness ratio of columns resisting moments to 20. Using the experimental formulas determined by Sáenz,† the factor of safety

*Mayer, R., *Die Knick Festing Keit*, Berlin, 1921, p. 139. Quoted by E. Butti, *Pandeo*, Buenos Aires, 1943, V. II, p. 174.

†Sáenz, Duplace, "Formulas Experimentales Para el Cálculo de Columnas de Hormigón Armado Cargadas Axialmente," Universidad de la Habana, 1952.

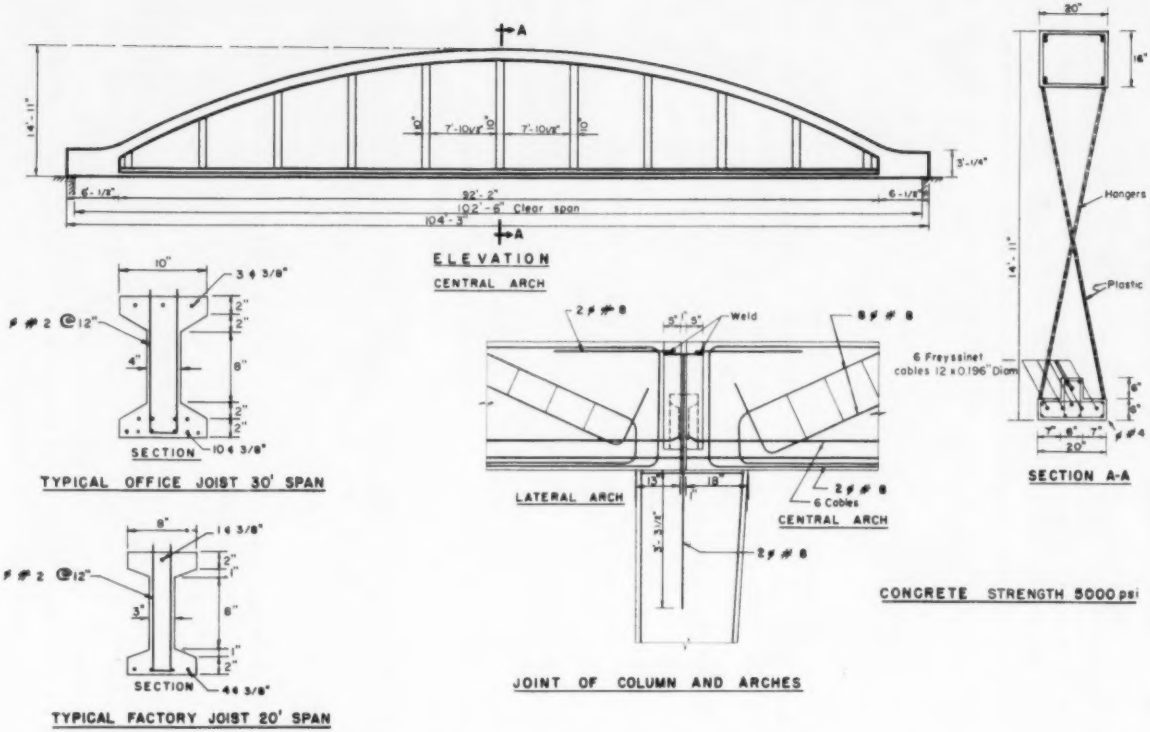

Fig. 2—Details of precast concrete units

against buckling was found to be 2.57. The reinforcement was determined by the formulas recommended for ultimate strength design by ACI-ASCE Committee 327, for members subjected to flexure and axial loads. In the direction of the vertical plane which contains the directrix of the arch the slenderness was reduced to insignificant values by the use of X-shaped hangers which are placed in vertical planes normal to the plane of the arch. Due to the vertical stiffness of upper and lower chords these hangers form a triangulated truss, which provides enough lateral resistance to avoid buckling.

Analysis showed that deflections in the horizontal tie subject to tension would be excessive if it were reinforced with common structural grade reinforcing bars. These deflections were unacceptable since they were principally produced by dead and live loads of the slab and joists, which would act after the arches had been placed in their final position. To reduce the working stress and thus diminish these deflections to make them acceptable involved uneconomical design, so it was decided to prestress the horizontal tie.

The prestressing force is taken up mostly by the concrete of the straight tie, and the subsequent superimposed loads produce loss of compression in the concrete with very small deflections. Because of the low stresses induced in the concrete, the resulting deflections were admissible. The prestressing force applied was such that if all the area bearing on the arch were fully loaded with the design live load, the stress in the concrete of the tie would be reduced to zero. This solution eliminates cracks in the concrete, thus offering better protection to the reinforcing steel.

The Freyssinet system was used for prestressing. As in this system it is not necessary to bond the prestressing steel, the strands were placed in plastic tubes of the same type as fabricated by this factory. After prestressing, grout was injected into the tubes. Using this method, compressive stresses due to the central strands counteracted the tension stresses produced by the tensioning of one of the outer strands.

Indentations were made on the projecting wires in the fixed end to check individual displacement and to correct the elongation measured in the tensioning end due to anchorage adjustments.

Fig. 2 shows details of reinforcement and dimensions of the 102 ft 6 in. arch, as well as the details of the joint between the arches and the columns. The prefabricated columns were tapered from a square 16 x 16-in. section at the bottom to a rectangular 16 x 32-in. section at the top to provide the necessary bearing area to support the arches.

To eliminate the fill needed to form the drainage slopes of the roof, the inside columns were made higher than those of the perimeter; reducing the total load to produce a more economical structure.

The roof was designed for a live load of 40 psf. This load, heavier than usual for roofs, was due to mechanical and electrical installations hung from it and shelving placed on top for weather testing of samples.

CONSTRUCTION

To obtain the most advantageous conditions for the owner, the plans and specifications given to the bidders included several alternates and the final details were worked out with the engineer of the successful bidder.

The contractor decided to use prefabrication in connection with prestressing as much as possible, and the only elements cast in place were the foundations and the slabs. The columns, beams, and joists were fab-

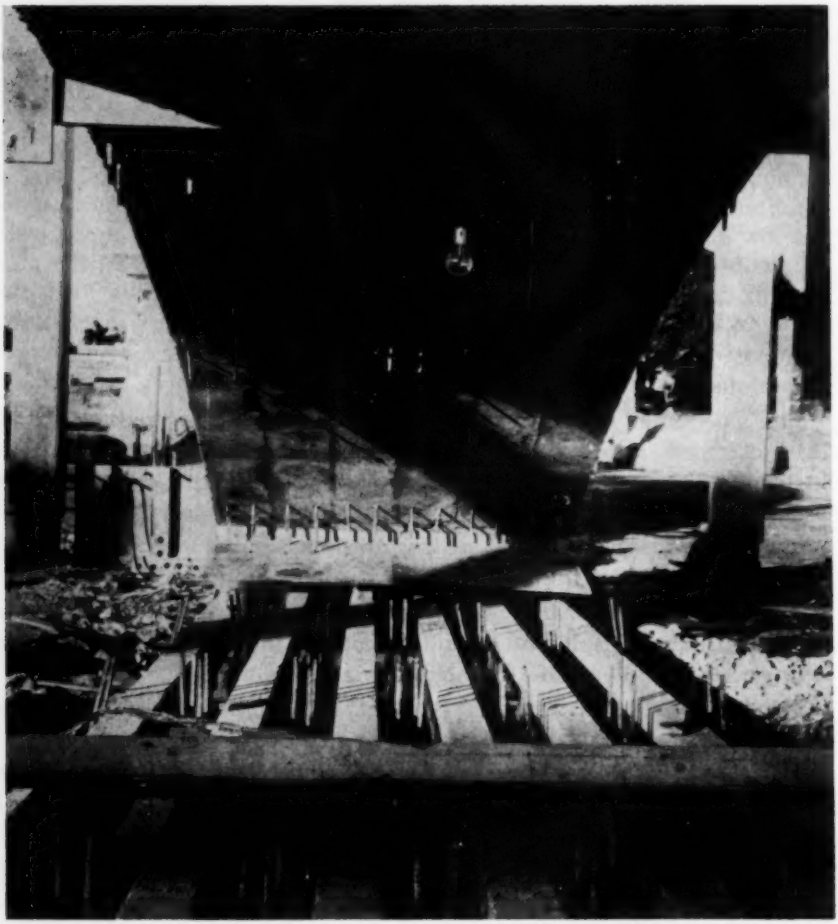

Fig. 3—Arches stripped of forms and ready for installation and welding of hangars

TABLE 1—UNIT COSTS

	Cost, per sq ft of useful area	Cost per sq ft of covered area including overhang	Cost per cu ft
Cover only	$ 1.57	$ 1.44	$ 0.083
Complete structure	2.05	1.88	0.108
Building	3.34	3.10	0.176

ricated at the contractor's plant but the arches were prefabricated on the site to eliminate transportation. First the ties and the hinges were concreted with projecting anchorage for the reinforcement of the arch and hangers (Fig. 3). Second, the arches were cast also with projecting anchorage for hangers. The anchorage of the hangers consisted of U-shaped rods with projecting ends which permitted butt welding. Due to the shortness of the hangers no special care was necessary to obtain an equal stress distribution on each rod.

After the concrete reached 3500 psi strength tensioning of strands was started, followed by grouting of the ducts. Since the strands are straight the friction loss was neglected and tensioning could be done from only one end. First the central strands were tensioned and then carried on symmetrically with the outer ones.

Fig. 4 and 5 show the erection of the arches and the assembly ready for casting the joints.

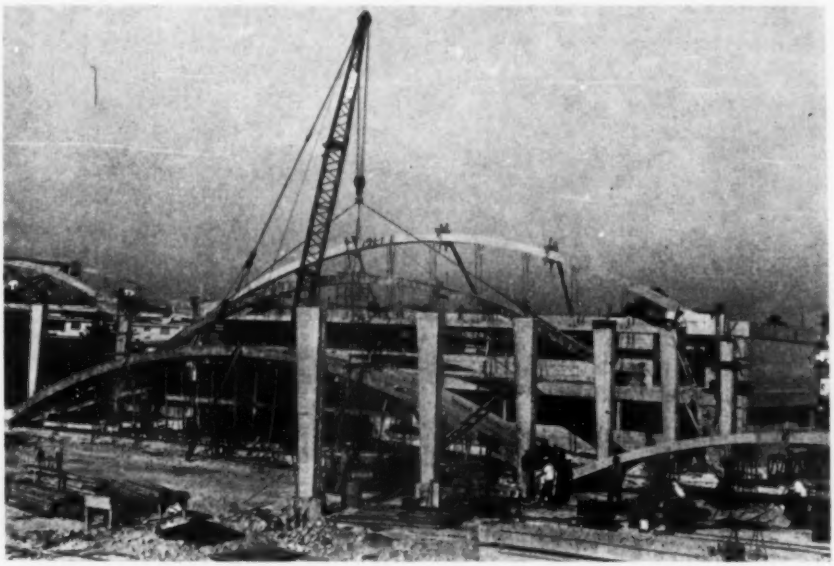

Fig. 4—Erecting arches on precast columns

COST ANALYSIS

The total cost of the project was $230,000. Of this amount approximately $62,000 covers the cost of the structure itself, $54,000 for the industrial area and $14,000 for the office area.

Table 1 shows unit costs for the cover only, the complete structure including the cover, and the total cost of the building including walls, floors, windows, etc.

If readers from other countries wish to make a more realistic evaluation of these figures, they should know some typical indexes of construction costs in Havana:

Foreman	1.20 US dollar per hr
Carpenter	0.80 US dollar per hr
Common labor	0.50 US dollar per hr
Portland cement	1.20 US dollar per bag (94 lb)
Stone, gravel	3.00 US dollar per cu yd
Mild steel bars	160.00 US dollar per ton
Lumber	0.16 US dollar per fbm

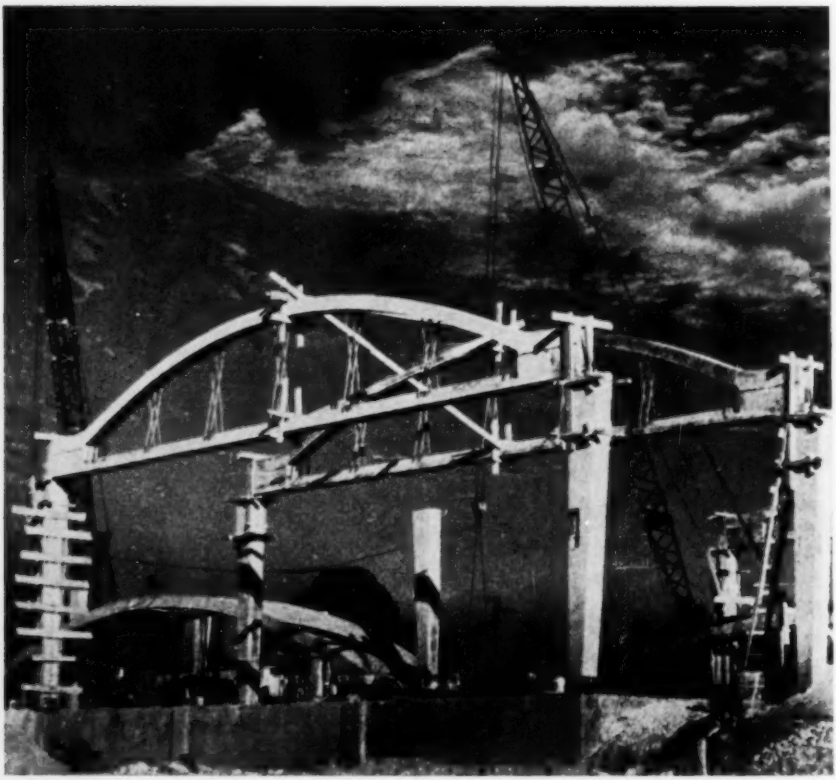

Fig. 5—Assembly ready for concreting arch-column joints. Temporary bracing to maintain position

CONCLUSION

The plant was opened early in January, 1958, and the owner was well satisfied with it. The authors feel that the building complies with all the requirements stipulated, and at a very satisfactory price.

Inmobiliaria Plincar S. A. is the owner; Sáenz-Cancio-Martín-Gutiérrez, Ingenieros-Arquitectos (SACMAG), Havana, Cuba, were in charge of design and supervision; and Fernando Munilla was the general contractor.

Based on material originally submitted to the Institute May 10, 1959. Presented at the 12th regional meeting of the Institute, Mexico City, Mexico, Nov. 3, 1959. Title No. 57-45 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 8, Feb. 1961 (Proceedings V. 57). Separate prints are available at 50 cents each.

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Discussion of this paper should reach ACI headquarters in triplicate by May 1, 1961, for publication in the Part 2, September 1961 JOURNAL

Title No. 57-46

Rheological Behavior of Hardened Cement Paste under Low Stresses

By JOSEPH GLUCKLICH and ORI ISHAI

Rheological experiments determined the complete rheological model of hardened cement paste. It is shown that for a *W/C* ratio of 0.32 and air-sealed material the model consists of: (a) a Hookean spring; (b) three Kelvin elements, of different parameters, in series; (c) a nonreversible Kelvin element; (d) an element comprising a dashpot, representing deformation due to microcracking. All these elements are connected in series. Elements (a) and (b) represent reversible deformation, while (c) represents a nonreversible deformation and (d) a small deformation reversible up to a few hours after loading and nonreversible thereafter. It is suggested that the deformational response of the material to load is dependent on hygrometric conditions to such a degree as to make the resulting quantitative differences seem like differences of quality. The various rheological parameters were determined.

■ IN A RECENT PAPER,[1] one of the authors described experiments to determine the rheological structure of hardened cement paste. These experiments provided positive indication that the material was a solid (and not a liquid), with instantaneous *linear* elasticity, reversible (primary) and also nonreversible (secondary) creep. Two important features, however, remained untreated in detail: permanent set* and the mechanism of the nonreversible creep. The former was measured in an experiment involving a considerable lapse of time between loading and unloading, and although strictly instantaneous deformations were measured in all cases, there was no way of telling whether there would be any permanent set in the case of immediate unloading. The existence of the latter (the nonreversible creep) was established in a series of alternating loading and unloading. However, since all cycles were of equal length, and thus resulted in the same amount of total creep, it could not be said conclusively whether or not the secondary creep is independent of the primary creep. Only by varying the total creep from one cycle to another would it be possible to isolate the two processes.

In addition to the clarification of the above points, it was considered important to study the observed erratic deformations[1] along the cir-

*By permanent set we denote an *instantaneous* nonreversible deformation as distinct from secondary creep, which is a *time-dependent* nonreversible deformation.

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cumference of a cylinder in axial compression, as it was felt that this phenomenon is somehow related to the mechanism of creep. The present paper accordingly reports on experiments conducted with a view to determining the entire rheological model of hardened cement paste.

TESTS

The beams used were the same as in the previous study, i.e., 83.5 x 2.27 x 2.27 cm (32.9 x 0.86 x 0.86 in.) cast from a cement paste of $W/C = 0.32$, the cement being an ordinary portland cement. On demolding (after 24 hr) the specimens were wrapped in a sheet of nylon to seal the material against the atmosphere.*

The specimens were prepared, stored, and subjected to loading at the age of 28 days under controlled conditions, the temperature being 16 C (61 F).

Test A — Permanent set

An apparatus (Fig. 1) similar in principle to the one described in the previous study[1] (except for a modification enabling the beams to be mounted vertically) was used for horizontal loading. The beams were simply supported on two knife-edge supports, and the load was applied at the center of each beam by a horizontal string, a pulley and a weight, thus eliminating any dead load effect and permitting no-load recovery. Center deflections were measured horizontally by a 10^{-4} in. dial gage.

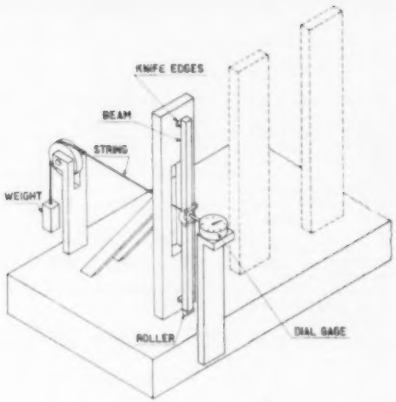
Fourteen beams were loaded in pairs with the following weights: 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, and 8.0 kg (4.41, 6.62, 8.82, 11.02, 13.24, 15.46 and 17.67 lb).

The procedure was as follows: A reading of the dial gage was noted, the respective load then applied and immediately removed, upon which the gage reading was again noted. No reading was taken while under load, so as to permit the reduction of loading time to a small fraction of a second. This procedure ensured elimination of any possible time effect, and the difference between the two readings noted, if any, would thus be the permanent set as defined. After that, a 4.0 kg (8.82 lb) load was applied to all beams, the instantaneous deformation noted, and the load left to act. The beams were successively unloaded at 1, 2, 5, 7, 8, 15, 19, 26, 32, and 37 weeks, and the instantaneous deformations noted. The difference, for each beam, between the deformations at loading and unloading gave the permanent set as affected by the time under load.†

*In the previous study,[1] the beams were coated with a polysterene solution for the same purpose. This formed a skin adhering to the specimen. The present method was only adopted for convenience, and the effect of this difference was not appreciated until later.

†This may seem a paradox in view of the footnote on p. 947, where permanent set was defined as a *time-independent* deformation. It is maintained, however, that in both cases (i. e., with and without a time interval between loading and unloading) the deformation in question is still an *instantaneous* one. The difference in the mechanisms of the two cases is explained in the first paragraph of "Discussion of Results," p. 952.

Fig. 1—Loading apparatus



Test B — Long-time creep

The 14 beams used in Series A were also tested for time deformation. Deflection readings under the 4.0 kg (8.82 lb) load were noted against time and the total creep curve was thus obtained. As mentioned for Series A, the beams were unloaded at varying intervals from 1 to 37 weeks, and left to recover under no load with deflection readings again noted against time. The recovered and nonrecovered components of the creep were thus obtained separately, and the asymptotic value of each determined as a function of the time under load.

Test C — Short-time creep

A test almost identical with Series B (except for the time under load being of the order of hours instead of weeks) was carried out on eight other beams, using the same apparatus. The times were: ½, 1, 2, 4, 8, 24 hr; 1, 3, and 5 days.

Test D — Extra-short time creep

To study more closely the erratic circumferential creep observed in the compression of cylindrical specimens, strain gage measurements were taken on the tension and compression surfaces of beams tested in flexure. Several beams of the same cross section as before, but much smaller in length, were clamped at one end and loaded at the other. The strain gages (SR-4, Type A-5) spaced as shown in the inset of Fig. 7, were connected to a bridge. A 2 kg (4.41 lb) load was applied and readings were noted against time, initially at short intervals and subsequently at gradually increasing ones. The load was allowed to act for 48 hr, and the same readings taken after 5 days' recovery.

RESULTS AND ANALYSIS

The results of Series A are presented graphically in Fig. 2, with the differences between instantaneous deformations at loading and unloading plotted against the time interval between them. As the graph shows, at zero time the deformations at loading and unloading were equal. This result was obtained for loads ranging from 2.0 to 8.0 kg

TABLE 1 — LONG-TIME CREEP

Time under load	Deformations, 10^{-4} in.									
	1 week	2 weeks	5 weeks	7 weeks	8 weeks	15 weeks	19 weeks	26 weeks	32 weeks	37 weeks
1. Instantaneous deformation on loading	271	258	267	262	279	254	289	260	249	259
2. Instantaneous deformation at end of loading	250	256	255	250	259	222	268	250	221	227
3. Total deformation at end of loading	439	493	635	628	654	648	854	806	776	830
4. Nonrecovered deformation after final recovery	29	62	187	176	205	234	360	370	380	410
5. = (1) − (2) Aging of instantaneous elasticity during time under load	21	2	12	12	20	32	21	10	28	32
6. = (4) − (5) Nonreversible creep	8	60	175	164	185	202	339	360	352	378
7. = (3) − [(1) + (6)] Reversible creep	160	176	193	202	190	192	226	186	175	193

TABLE 2 — SHORT-TIME CREEP

Time under load	Deformations, 10^{-4} in.							
	½ hr	1 hr	2 hr	4 hr	8 hr	24 hr	3 days	5 days
1. Instantaneous deformation on loading	262	233	258	268	277	250	242	248
2. Instantaneous deformation at end of loading	258	229	249	260	262	249	237	230
3. Total deformation at end of loading	286	264	290	315	332	334	357	381
4. Nonrecovered deformation after final recovery	0	3	4	11	13	21	15	50
5. = (1) − (2) Aging of instantaneous elasticity during time under load	4	4	9	8	15	1	5	18
6. = (4) − (5) Nonreversible creep	0	0	0	3	0	20	10	32
7. = (3) − [(1) + (6)] Reversible creep	28	32	37	44	57	64	105	101

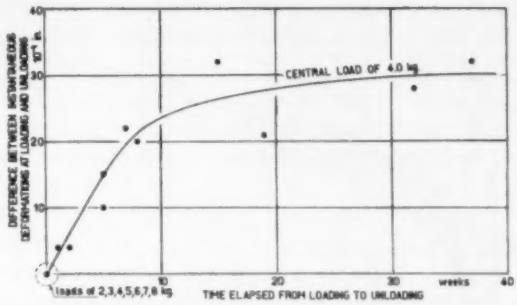


Fig. 2—Difference between instantaneous deformation on loading and unloading versus time under load

(4.41 to 17.67 lb); the rest of the curve refers to a 4.0 kg (8.82 lb) load only.

Fig. 3 represents the results of Series B. It shows the behavior of 14 beams under a central load of 4.0 kg (8.82 lb), and their respective recoveries for different loading times, with sufficiently long recovery periods allowed so as to reach practically asymptotic values. This permitted quantitative differentiation between the reversible and the nonreversible creeps. It should, however, be borne in mind, that the nonrecoverable component includes the gain of the *instantaneous* deformation (due to the latter being smaller on unloading than on loading) which should be excluded when strictly *time deformations* are considered. Accordingly, the following procedure was adopted for determining the various components of the total deformation:

(a) The total nonrecoverable component was measured as the asymptote of the recovery.

(b) The gain of the instantaneous deformation was calculated as its difference between loading and unloading.

(c) The pure nonreversible creep was calculated as (a) minus (b).

(d) The reversible creep was calculated by subtracting (a) and the instantaneous recovery from the over-all deformation just before unloading.

The values obtained by this procedure are given in Table 1, and the reversible and nonreversible creep [(d) and (c) respectively] are plotted against the time under load in Fig. 4.

The results of Series C (the short-term creep) are presented in Table 2. (The actual curves for deflections and recoveries, being of the same character as in Fig. 3, are not presented here.) In preparing this table the same procedure was followed as in the case of the long-term creep; and here too the reversible and nonreversible creeps are plotted against time in Fig. 5.

Fig. 6 represents the strain gage readings of two beams loaded as cantilevers. The respective points of measurement were located in pairs under identical conditions (gages 5-9, 6-10, 7-11, 8-12), and the readings plotted accordingly, differences within the pairs being apparently due

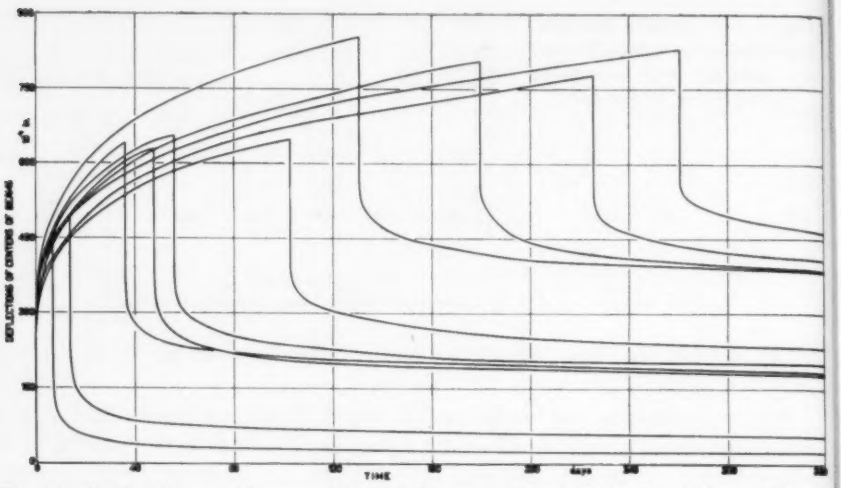


Fig. 3—Deflections and recoveries of beams subjected to load for different periods of time

to statistical scatter. Gages 5, 7, 9, and 11 were mounted on the tension surface and gages 6, 8, 10, and 12 on the compression surface. The bending moments were: for points 5, 6, 9, and 10—47.0 kg-cm (40.9 in.-lb) and for 7, 8, 11, and 12—31.6 kg-cm (27.5 in.-lb). Extensions are indicated as positive and contractions as negative. The graph is confined to a 2-hr interval to save space, the strains beyond this limit proceeding in a fairly monotonic manner right up to the removal of load, and the recoveries—to the end of the test. It should be borne in mind that the initial points of the recovery graphs, represented by the coordinates of zero time and zero strain, are arbitrary.

DISCUSSION OF RESULTS

Permanent set

In the paper referred to earlier,[1] permanent set was defined as the time-independent nonreversible deformation. Since in that study the resulting instantaneous deformation on unloading had been found smaller than its counterpart on loading, it had been assumed that the difference between these two values was a permanent set. However, there had been doubt as to whether there would be such a difference on removing the load a fraction of a second after application. It was considered that the local fractures which constitute this type of deformation (after Reiner[2]) might in fact set in instantaneously with loading, but with the time interval too short the cracks close on unloading so as to render the entire deformation reversible. If, however, some time elapses before unloading, increased hydration, due to intensified ion movement, may deposit its products on the newly formed surfaces and

prevent complete recovery. In such a mechanism permanent set will set in *only* with an adequate time interval between loading and unloading.

This point was studied in the present work. From Fig. 2 it is immediately evident that there is no permanent set for a zero time interval, this was verified for loads approaching the breaking load (for the beams in question, approximately 10 kg or 22.1 lb). For finite time intervals a progressively increasing permanent set was observed; it still remained to be decided, however, whether this was indeed a permenent set as defined in the preceding paragraph, or merely an increase in rigidity which one would naturally associate with aging. For this, two groups of beams, identical with the ones in question and under identical conditions, were tested for rigidity at increasing ages. One group was subjected to a concentrated load of 4.0 kg (8.82 lb) on the apparatus shown in Fig. 1, the loads being removed every few days, and immediately reapplied, and the instantaneous deformational response observed. The other group was tested on the same apparatus under no-load conditions, but the load of 4.0 kg (8.82 lb) was applied every few days, and immediately removed, the instantaneous responses being noted. (The study of this group is still in progress.)

The results of both groups are shown in Fig. 7. Under such loading procedures as in these two cases the decrease of deformation can be attributed to nothing but aging, and since in Group (a) the time decrease of the deformation is in fair agreement with the rise in the difference between deformations in Fig. 2 (indicated by a dashed line

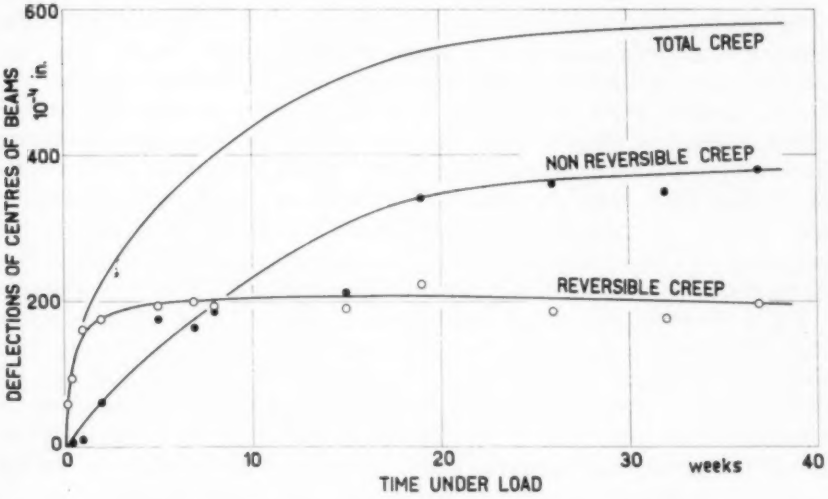


Fig. 4—Creep components versus time under load (for long-time creep)

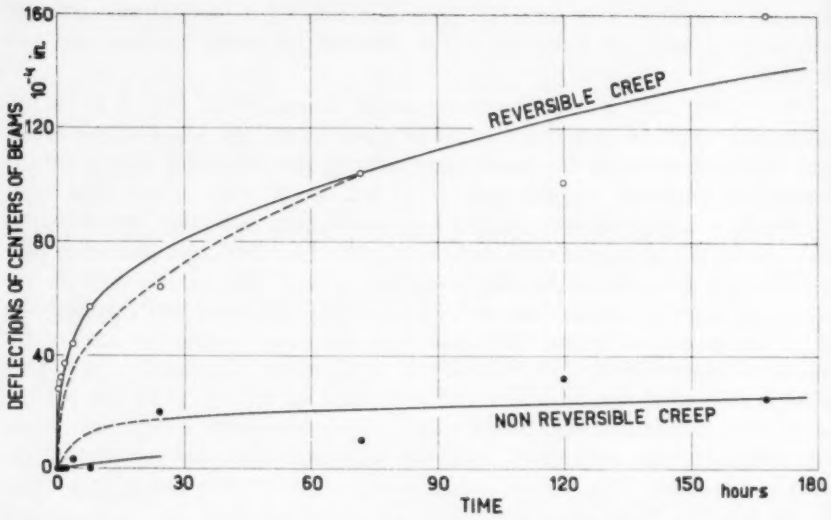


Fig. 5—Creep components versus time under load (for short-time creep)

in Fig. 7), we can safely conclude that the latter is not a permanent set but the increase in rigidity due to aging. Thus permanent set is definitely absent in hardened cement paste of the W/C in question under conditions as described.

Creep

As pointed out earlier, one object of the present work was to determine whether the two types of creep (viz., the reversible and the nonreversible) are two components of a single mechanism (in which case they should have the same retardation time) or two distinct mechanisms. In the present work the two types of creep were isolated for *various times under load*. Fig. 4 shows that the reversible creep has practically ended after 10 weeks whereas the other is still increasing at 40 weeks. It is thus clear that the two deformations are entirely independent, and cannot be represented by means of a single Kelvin element as suggested in Fig. 11 of Reference 1, but rather by means of two elements, as suggested in Fig. 10 of the same work,[1] and in Fig 8 of this paper.

Fig. 5 shows these two deformations as produced by short times under load. One feature of this graph deserves attention: for up to 8 hr loading there is almost no nonreversible creep, which means that the over-all deformation is reversible; for 24 hr loading the nonreversible creep is already of the order of 20×10^{-4} in. The rate of increase beyond 24 hr would not justify such a jump, and it seems as if an initially reversible deformation became nonreversible between the above periods. Indeed, if the curve of the nonreversible creep is produced towards $t = 0$ in a

gradual manner (broken line), and the intercept between it and the experimental curve subtracted from that of the reversible creep, the two curves become more uniform. This point will be referred to again later.

Extra-short time creep

From Fig. 6, where only typical results are shown, we learn that during the 30 min or so after application of the load some internal readjustment takes place. At points 5-6 and 9-10 the behavior was as

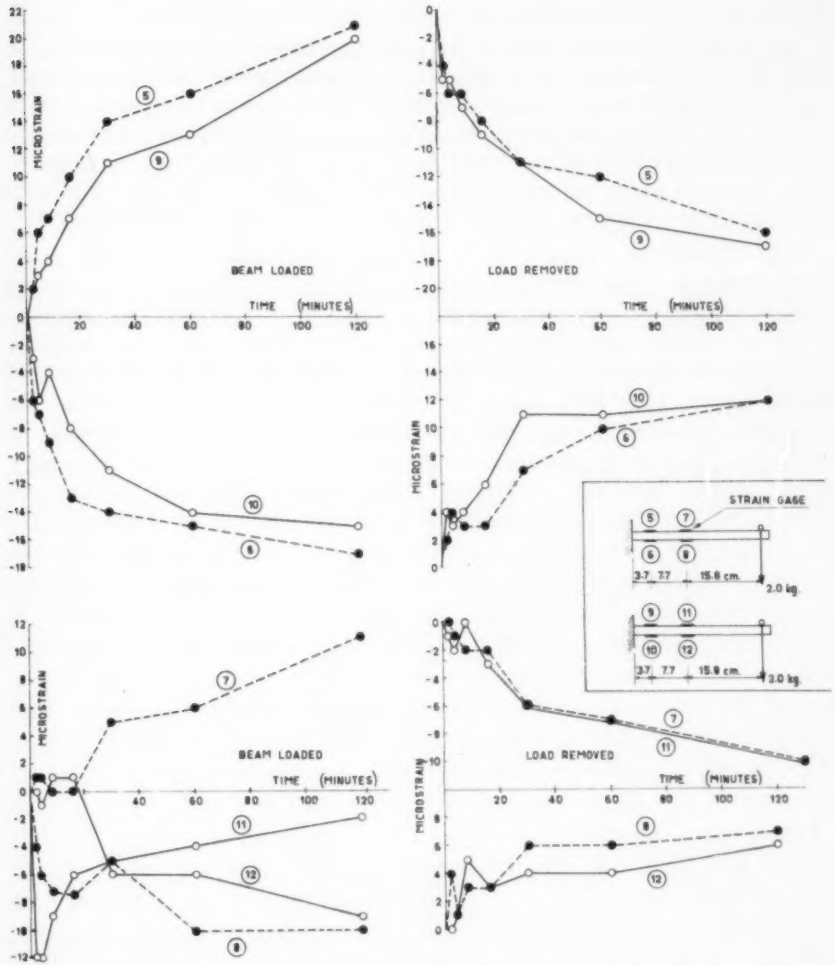


Fig. 6—Local strains in beams versus time measured by electrical resistance strain gages

would be expected. At points 7-8 there is already some evidence of a noncontinuous process, and at points 11-12 a sudden development set in about 2 min after application of the load. Readings of Gage 11 show contraction instead of extension. The contraction apparently took place soon after loading, and after that the deformation progressed quite normally, although not large enough to become an extension. Results obtained from other gages, not presented here, show similar behavior in some cases, i.e., an extension which becomes contraction, and in other cases an extension which suddenly becomes much larger; the over-all deformation as measured by a dial gage at the free end of the cantilever does not indicate any such erratic behavior.

The explanation the authors suggest for these phenomena has to do with the spontaneous development of micro-cracks. Macroscopically, the formation of statistically dispersed micro-cracks is observed as a continuous deformation; however, when time measurements are carried out with the aid of electrical resistance gages, a crack forming immediately below a gage, causes it to record a sudden extension, while one forming close to a gage may be recorded as a sudden contraction.

It is difficult to estimate these sudden deformations from the strain gage readings. The latter are reliable only when the entire surface in contact with the gage participates in a uniform deformation, while in a case of concentrated deformation the strain measured is considerably exaggerated. However, taking the vertical interval between Curves 7 and 11 (about 10×10^{-6}) as a very rough approximation, and referring it to the gage length of 1 cm, we obtain an absolute displacement of 10^{-5} cm. This crude calculation shows that the cracks are of microscopic order. They are not the sort of cracks which lead to fracture, rather the opposite, they eliminate potential centers of fracture, thus permitting the material to adjust itself to the state of stress. The behavior is similar to that of a plastic material where peaks of stress are relieved by local plastic yields. In the present case the material is brittle and local high peaks of stress can only be relieved by transformation of elastic energy into surface energy, or in other words, by expansion of the cracks. In an ideally homogeneous material this should lead to total fracture, since the mechanism involved is one of unstable equilibrium; in real materials, on the other hand, the cracks are soon stabilized by zones of low elastic energy or high surface energy.*

Considering the time-dependence of the process, it is quite possible that at a W/C of 0.32, the air-sealed specimen is in a state of almost total capillary saturation, so that the transmission of the external load to the solid phase involves a time lag, and micro-cracking develops gradually with time. In air exposed specimens, or at a different W/C,

*A total fracture may ensue also in real materials under conditions of unlimited potential energy in the system.

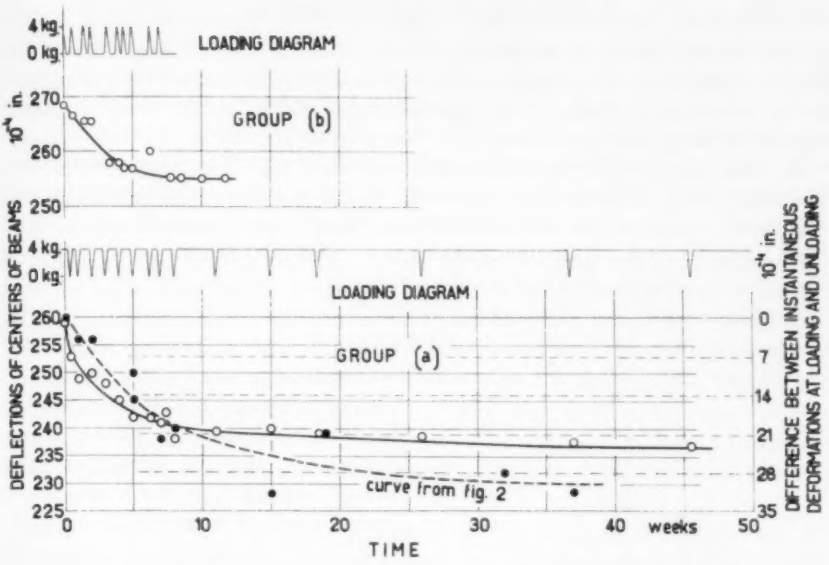


Fig. 7—Effect of age on instantaneous deformations. Group (a)—Repeated loadings of unloaded beams. Group (b)—Repeated unloadings of loaded beams

with partial capillary saturation, this deformation, or part of it, may be instantaneous, in which case the material will have a permanent set, for the mechanism described here is the same as that which causes permanent set, except for the time lag. This cracking deformation may indeed be defined as delayed permanent set. Since the time interval involved is of the order of 30 min, it is reasonable to assume that the flow in question is capillary, in contradistinction to the other deformations involving flow in the gel pores. The authors suspect that in many cases, owing to the insufficient accuracy of the measurement technique, this delayed set was mistaken for an instantaneous set which, as was shown, is completely absent in the present case.

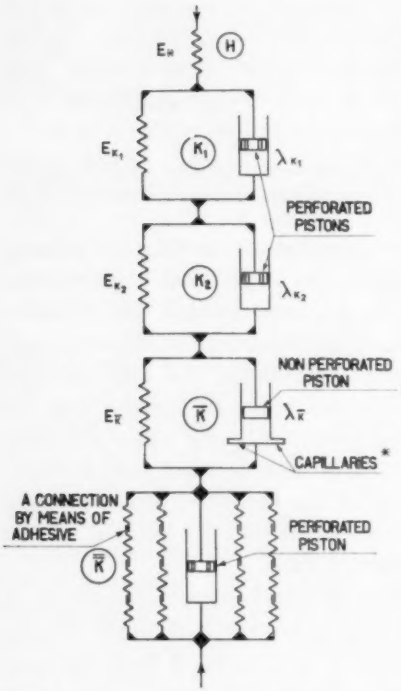
It seems that since as shown in Fig. 5, the entire deformation is reversible up to 8 hr loading, the cracking deformation must also be reversible for such times, and only after a day or so it becomes nonreversible. In fact it is this deformation which had been stated to be initially reversible, becoming nonreversible later. Such behavior can be explained as follows: After a short loading the cracks can close almost completely on removal of load, but when the material remains a longer time under load hydration deposits may prevent a total closure. In the first case the deformation is reversible; in the second, there is a nonreversible part.

THE RHEOLOGICAL MODEL

The model proposed is shown in Fig. 8. Element H represents the entire instantaneous response which is elastic and linear, its parameter being stress-independent and age-dependent. The other four elements represent time deformations.

K_1 and K_2 are Kelvin elements representing the reversible creep (actually delayed elasticity). As will be shown later, the need for two elements is due to the two distinct retardation time factors involved.*

$\overline{K}$ represents the nonreversible creep. In the model this element is rendered nonreversible by means of capillary tubes providing a head exceeding the maximum possible force in the parallel spring. The nonreversibility of this component is due to the following processes: (a) Water diverted from saturated to under-saturated zones is firmly bound there by the high adhesion forces augmented by the enormous surface of the gel structure. (b) Water expelled to the atmosphere under the external load, or evaporated due to a gradient of humidity between specimens and atmosphere, is not recovered on removal of the load. This precludes the restoration of the material to its original shape as meniscuses are formed at the free ends of the channels. (c) Water displaced by the external load takes part in increased hydration, i.e., in a nonreversible chemical reaction, as new ionic balances are formed (Freyssinet[3]).



*Diameter of capillaries is such that they provide a head exceeding the maximum possible force in the parallel spring.

Fig. 8—Modified rheological model for hardened cement paste

K represents two kinds of deformation: (a) the cracking deformation or delayed permanent set, and (b) a component of delayed elasticity in addition to K_1 and K_2. The springs in this element consist of separate links held together by adhesive. The model thus comprises an almost infinite number of links varying in strength, in statistical distribution, from zero to a certain maximum. As the load is trans-

*Strictly speaking, the reversible creep comprises a third retardation factor of extremely small order. However, as shown further on, this factor is embodied in the $\overline{\overline{K}}$ element.

mitted from the dashpot to the springs, the latter yield and the links snap successively as the stress reaches the corresponding level of strength. Owing to the large number of chains of springs in parallel, some remain intact even under very high loads, providing the ultimate strength of the specimen is not exceeded. These residual chains of springs restore the element to its original shape upon removal of the load. However, under prolonged loading the loose ends of the ruptured springs coalesce,* and part of the deformation remains unrecovered on unloading. The snapping of the links represents the spontaneous expansion of cracks as reflected in the erratic performance of the strain gages. Thus, this element describes a deformation which is reversible under a short-time loading and nonreversible under prolonged loading.

The other deformation represented by the K element, viz., the third component of delayed elasticity, is simply due to the springs (as distinct from the links) in parallel with the dashpot. The retardation time of this component is much smaller than that of the other two K elements.

As to the deformation due to cracking, further research is called for before any quantitative conclusions can be stated. It is, however, likely, that under certain conditions of humidity the model will comprise one more element similar to $\overline{\overline{K}}$, but without a dashpot, representing the instantaneous permanent set.

THE RHEOLOGICAL PARAMETERS

Owing to the nature of the test, the parameters determined in this work were not concerned with pure distortion but rather with normal stresses. No information is available with regard to Poisson's ratio (which had been determined in a previous study[1] for instantaneous elasticity, but not for the various creeps), and consequently the coefficients of distortion are not known.

Referring to the model in Fig. 8, the over-all deformation under a constant load can be expressed as a function of time as follows:

$$\varepsilon_{n(t)} = \frac{p_n}{E_H} + \frac{p_n}{E_{K_1}}\left[1 - \exp\left(-t/T_{K_1}\right)\right] + \frac{p_n}{E_{K_2}}\left[1 - \exp\left(-t/T_{K_2}\right)\right] + \frac{p_n}{E_{\overline{K}}}\left[1 - \exp\left(-t/T_{\overline{K}}\right)\right] + \frac{p_n}{E_{\overline{\overline{K}}}}\left[1 - \exp\left(-t/T_{\overline{\overline{K}}}\right)\right] \quad (1)$$

where $\varepsilon_{n(t)}$ = over-all normal deformation after time t
p_n = normal traction
T = λ/E = retardation time of Kelvin element
λ = coefficient of viscosity for normal traction (after Reiner[2])
E = Young's modulus

*In a process similar to healing.

Actually, there should be a sixth term representing the cracking deformation, but in view of its smallness (less than 10×10^{-4} in. according to Fig. 6), and lack of information as to its dependence on time and load, it is neglected here. On removal of the load the recovery* will consist of the first, second, third, and fifth terms, the fourth and sixth being nonreversible.

To use Eq. (1) for predicting deformations, the following nine parameters must be determined: E_H, E_{K_1}, λ_{K_1}, E_{K_2}, λ_{K_2}, E_R, λ_R, E_F and λ_F.

From the deflections versus time curve for the nonreversible creep in Fig. 4, a curve of deflection versus time-derivatives of deflection was constructed. Denoting the deflection by δ, and assuming an exponential relationship we have:

$$\delta = \delta_\infty(1 - e^{-t/T}) = \delta_\infty - \delta_\infty e^{-t/T} \quad \text{.....(2)}$$

$$\frac{d\delta}{dt} = \dot{\delta} = \frac{1}{T}\,\delta_\infty\, e^{-t/T}$$

$$\delta = \delta_\infty - T\dot{\delta} \quad \text{.....(3)}$$

If on plotting δ against $\dot{\delta}$ a straight line is obtained, its slope would be a measure of the retardation time, and the δ intercept will be δ_x; the value accordingly found for δ_x was 400×10^{-4} in. The asymptotic values of both curves in Fig. 4 being known, the difference $\delta_x - \delta$ was plotted against time on semilog paper in Fig. 9. If the functions were exponential then this would result in straight lines, for from Eq. (2)

$$\delta_\infty - \delta = \delta_\infty e^{-t/T}$$

$$\log(\delta_\infty - \delta) = \log \delta_\infty - \frac{t}{T} \log e \quad \text{.....(4)}$$

Indeed the nonreversible creep resulted in a straight line. The slope of this yielded T_R (which was a check for the method of derivatives mentioned before). E_F was determined from the equation for elastic deflection:

$$\delta_\infty = \frac{PL^3}{48\,EI} \quad \text{.....(5)}$$

and, λ_F from

$$T = \frac{\lambda}{E} \quad \text{.....(6)}$$

The semilog graph of the reversible creep, however, only became rectilinear after 2 weeks, which suggests exponential behavior only beyond this limit. The rectilinear interval was extrapolated to $t = 0$, and the $\delta_x - \delta$ intercepts between the original curve and the extrapola-

*Strictly speaking, distinction should be made between recoveries after short-time loading and recoveries after prolonged ones, as explained earlier.

tion line, plotted against time, yielded another straight line. The first line thus represented one Kelvin element K_1, with a retardation time of T_{K_1}, and the second line represented K_2, with a retardation time $T_{K_2} < T_{K_1}$. As in the case of $\overline{K}$, the values E_{K_1}, E_{K_2}, λ_{K_1}, and λ_{K_2} were determined from the two straight lines by means of Eq. (5) and (6).

With the described procedure carried out with extreme precision, a third rectilinear component, with a very short time of retardation, could be observed. This corresponded to the $\overline{\overline{K}}$ element, for which only the parameter $T_{\overline{\overline{K}}}$ was sufficiently distinct to be determined.

E_H was determined directly from Eq. (5), where $\delta = \delta_\infty$ is the instantaneous deflection upon application or removal of the load. Information with regard to the age dependence of this parameter is available but not presented here.

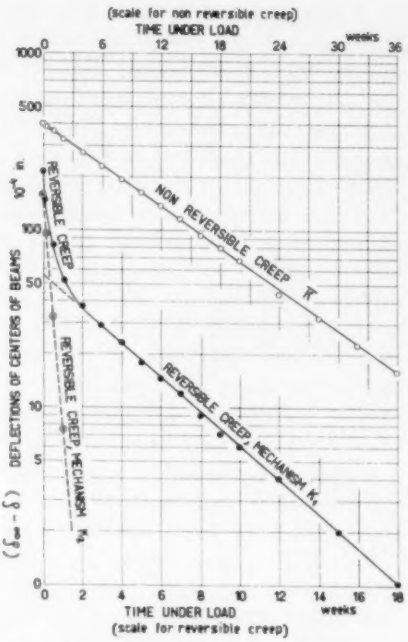


Fig. 9—Creep components versus time (for determination of parameters)

The values of the various parameters are summarized in Table 3.

DISCUSSION OF OTHER STUDIES

In the main, the results of this work sustain the theories put forward by the first author in his earlier paper,[1] with the following qualifications: (a) It was positively established here that at the degree of saturation in question, the material has no permanent set. (b) On the other hand, it was shown that part of the nonreversible creep was due

TABLE 3 — PARAMETERS

Element	Young's modulus E		Retardation time T, hr	Coefficient of viscous traction λ	
	kg per sq cm	psi		kg sec per sq cm	lb sec per sq in.
H	2.47×10^5	3.53×10^6			
K_1	11.8×10^5	16.9×10^6	763	32.4×10^{11}	46.4×10^{12}
K_2	4.14×10^5	5.91×10^6	56	0.8×10^{11}	1.14×10^{12}
$\overline{K}$	1.63×10^5	2.33×10^6	1860	10.9×10^{11}	15.6×10^{12}
$\overline{\overline{K}}$			$\approx 1/3$		

to a *delayed* permanent set. (c) The creep curves showed that the reversible creep comprises three or more (according to the degree of approximation required) Kelvin elements in series. (d) It was shown that the reversible and nonreversible creeps are two distinct mechanisms.

Accordingly, the model proposed in the earlier paper (Fig. 10), Reference 1) was modified in Fig. 8 of this paper. Apart from that, an additional discrepancy in behavior should be mentioned. While in the former case the nonreversible creep was smaller and of shorter retardation time than the reversible creep, in the present case it was the other way round. This difference, which is only quantitative, is due to the different air-sealing methods used (see p. 958). In specimens coated with polysterene, a skin adhering to the specimen was formed, and thus surface migration of the water was excluded altogether, whereas the nylon sheet wrapping still provided enough free space for water to wet the outside surface of the specimen and the inside of the sheet. The latter method, therefore favored a higher water loss, and hence a higher nonreversible creep, whereas in the former case only water diverted to to unsaturated zones or participating in renewed hydration was prevented from reversing on unloading.

This high sensitivity to hygrometric conditions is responsible for the diversity of opinions with regard to the nature of creep. Freudenthal and Roll,[4] for example, kept their cylinders exposed at 60 percent relative humidity, and in spite of corrections for shrinkage (measured on no-load control cylinders) their results were still much higher than would have been under conditions of totally excluded shrinkage. In fact, it had already been shown (by L'Hermite,[5] Neville,[6] and others) that creep and shrinkage are interdependent, and that the shrinkage of a loaded cylinder is higher than that of a control cylinder. It is quite probable that Freudenthal's curves of periodic loadings and unloadings (Fig. 6, Reference 4) would not have shown a continuous increase in creep from cycle to cycle, had shrinkage been excluded in his experiments. Instead, he might have found equal creeps at the end of each loading phase, as observed by the first author (Fig. 3, Reference 1). In such a case he might have abandoned his theory of concrete being a a liquid and would not have included a free Newtonian element in his model. Indeed, in the model proposed by the authors(Fig. 8), if evaporation were added to the mechanical load, suction of the piston in the $\overline{K}$ element would have ensued and the resulting curve would be similar to that reported by Freudenthal.

Furthermore, those workers who did not seal their specimens observed very small creep recoveries. In other words, the ratio of reversible to nonreversible creep was low in such cases. This, again is due to evaporation: water driven to the surface by the external load evaporates, and cannot be recovered on removal of the load. In a sealed specimen, on the other hand, a larger proportion of the water is recoverable, and

the above ratio would be higher. Generally, as one progresses from the extreme of total exposure to the opposite extreme of total submersion, higher values are obtained for this ratio. The two cases of nylon sheet and polysterene film are merely two intermediate stages in this case, the ratios being 0.5 for the first case, and 2.6 for the second. L'Hermite (Fig. 31 of Reference 5) reports a case of hardened cement paste specimens exposed at a relative humidity of 50 percent and a ratio of 0.25.

The immediate result of the above was that workers conducting their experiments under dry conditions tended to look upon concrete as a liquid, whereas those who worked under wet conditions tended to regard it as an elastic solid. The authors are planning further tests under different well defined conditions, including the two extremes, before forming their final opinion. In particular, it should be established whether underwater loading would result in nonreversible creep, apart from the small amount due to delayed permanent set. Some investigators (among them Mamillan,[7] who compared creep in air and in water) carried out their tests on submerged specimens, but unfortunately, did not attempt creep recovery on these specimens, so that while being able to show that creep under water is only a small fraction of creep in air (thus corroborating the various theories of seepage as the cause of creep), they were unable to state whether this creep was reversible or not. The point of whether there is a free Newtonian element in the model or not will be settled when the nature of this creep is established. However, all the evidence (in particular Fig. 3 of Reference 1) seems to point to the absence of such an element.

The fact that the nonreversible creep behaves exponentially in time (Fig. 4 and 9) and as a result never exceeds a certain limit, also contradicts the theory of free viscous flow.

ACKNOWLEDGMENTS

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Title No. 57-47

Ultimate Strength of a Folded Plate Structure

By GREGORY P. CHACOS and JOHN B. SCALZI

Objectives of this investigation were to determine the behavior of the folded plate structure as a simple beam with an irregular cross section and to verify the ultimate moment capacity by the rectangular stress block method. For this type of structure the ultimate collapse load agreed with the theoretical load within 1.8 percent.

■ A LONG, SIMPLY SUPPORTED FOLDED PLATE model is investigated for ultimate load capacity based on the action of a simple beam. The predicted ultimate moment capacity by the rectangular stress block method is compared with the ultimate moment developed in a reinforced mortar structure loaded to failure.

The test structure was loaded with bricks on the top horizontal plate. Failure occurred when the tensile reinforcement ruptured. The general action of the structure at failure indicated that the simple beam assumption is valid for the type of structure tested. The ultimate load on the model was 490 lb per sq ft on the loaded plate.

The objectives of the investigation were to determine the behavior of the folded plate structure as a simple beam with an irregular cross section and to verify the ultimate moment capacity by the rectangular stress block method.

TEST STRUCTURE

A three-plate configuration was selected as a typical cross section for a folded plate roof (Fig. 1). The thickness of the plates was established at ½ in. because this was the minimum which could be used to cover the reinforcement properly and develop sufficient moment for the transverse loads.

The individual plates of the structure were proportioned on the basis of the one-way slab action of the top plate in resisting the transverse bending moments. A width of plate of 12 in. was selected on the basis of minimizing local buckling effects. The width-to-thickness ratio of 24 resulted in an elastic buckling stress much greater than the ultimate compressive strength of the concrete. The length of the structure was

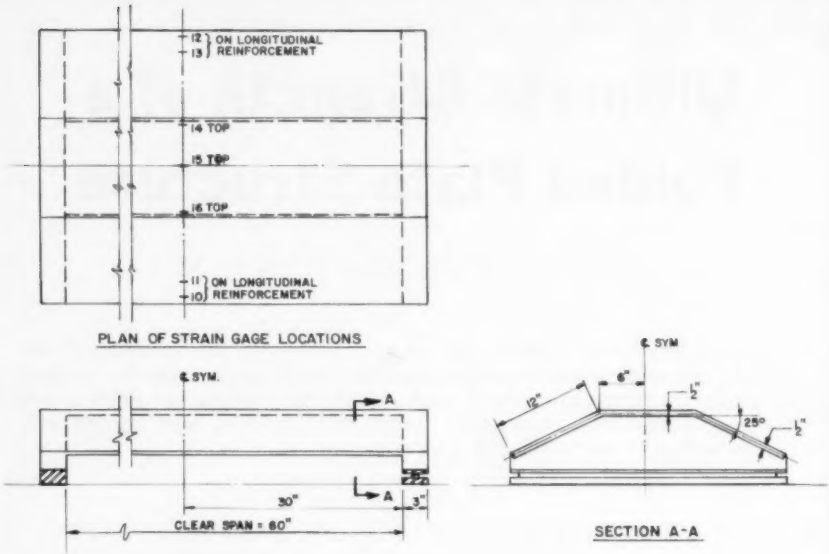

Fig. 1—Dimensions of the structure and locations of SR-4 strain gages

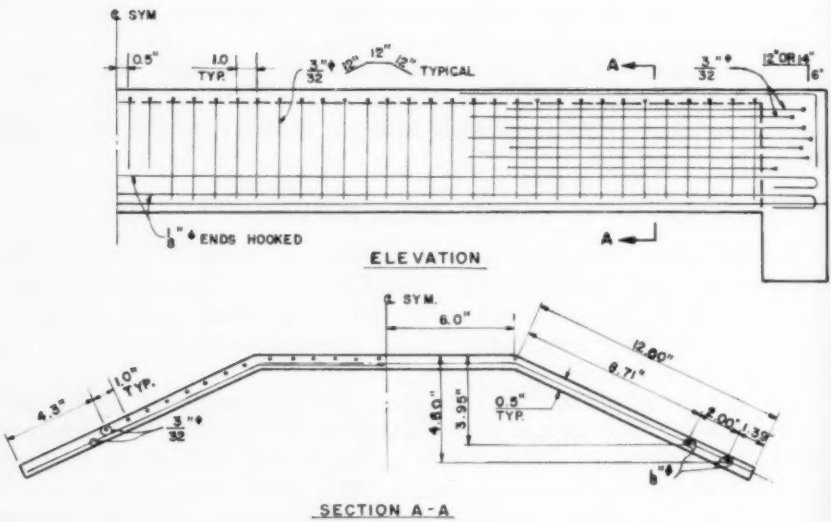

Fig. 2—Details of reinforcement

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ACI member **John B. Scalzi**, structural engineer, U.S. Steel Corp., Pittsburgh, was formerly an associate professor of structural engineering, Case Institute of Technology. Holder of a DSc in civil engineering, he has for years been an active consulting engineer.

set at 60 in. making a length-to-width ratio of 5 which is considered reasonable for folded plate action.

The angle of inclination and length of the side plates were determined on the basis of having the ⅛ in. diameter steel rods rupture before the concrete failed in compression.

The reinforcement was ⅛ in. in diameter, C 1213, screw machine steel in cold drawn rods which had been grit blasted to develop bond resistance (Fig. 2). The reinforcement for transverse bending, diagonal tension, and end effects were deliberately conservative to preclude the possibility of secondary failures.

The concrete mixture was a high-early-strength cement and sand which had an ultimate compressive strength of 1500 psi at the time of the test.

SR-4 strain gages were placed on the steel and concrete at midspan (Fig. 1). Three Ames dials were used to measure deflections (Fig. 3). One was located at the longitudinal and transverse center lines to measure the vertical movement, and the other two were arranged to measure the movement of one side plate.

The structure was simply supported along the full length of the end diaphragms.

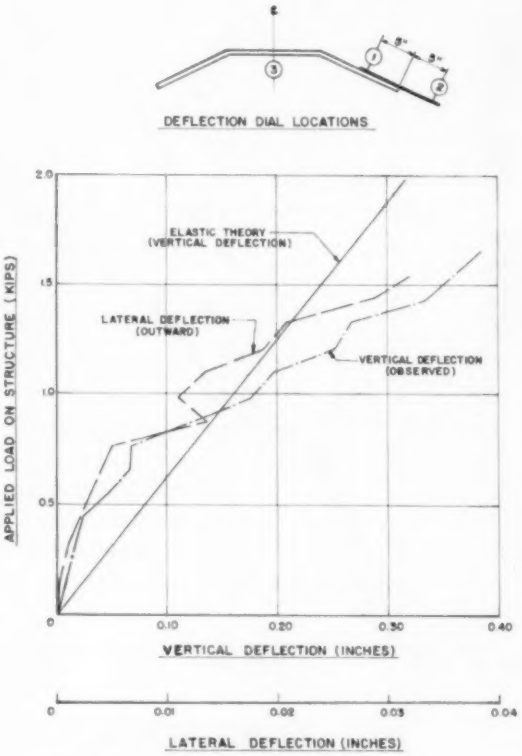

Fig. 3—Applied load versus deflections

ANALYSIS

The test structure was designed as an under-reinforced beam using the Whitney method for the ultimate strength determination. The center of gravity of the four rods used for longitudinal reinforcement was 4.375 in. below the top surface of the top plate. Using an ultimate tensile stress of steel of 106,000 psi, determined by test, the computed moment capacity was 21,900 in.-lb. The depth of the compression zone was less than the thickness of the top plate.

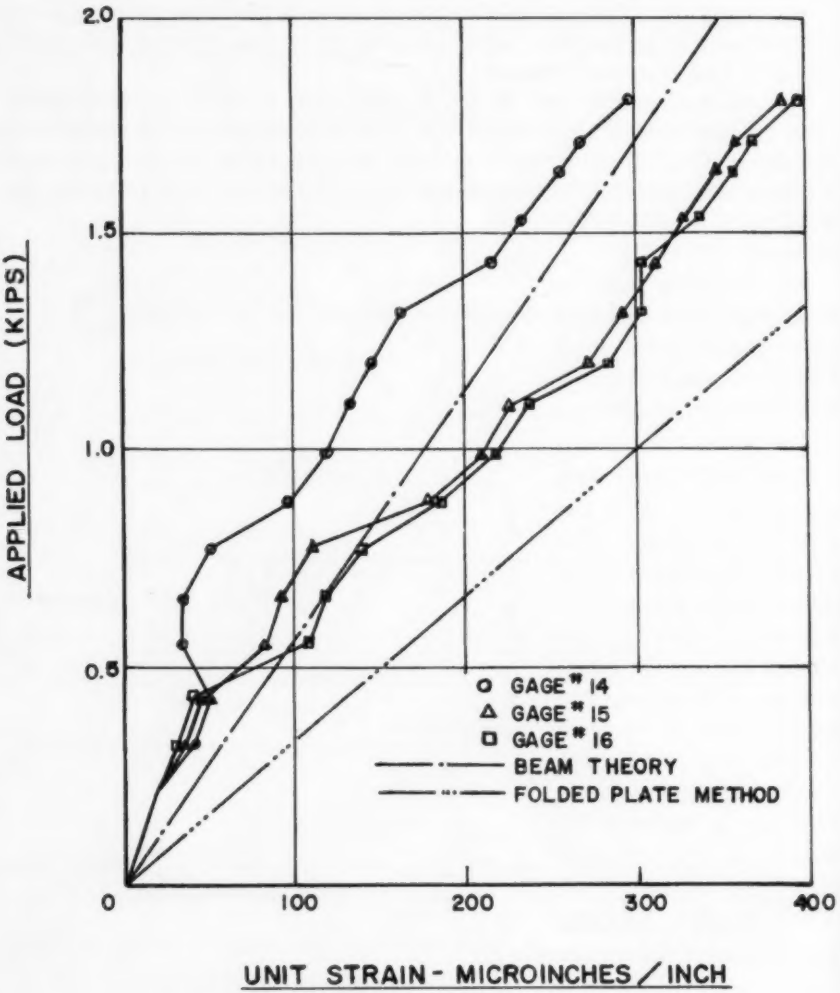

Fig. 4—Applied load versus strains on top surface of structure

TEST PROCEDURE

The load, applied to the top plate only, consisted of bricks placed on a 3-in. cushion of sand. The bricks were added in increments of 25 (approximately 111 lb) after which strain and deflection readings were recorded. The bricks were spaced to avoid interference with each other as the structure deflected.

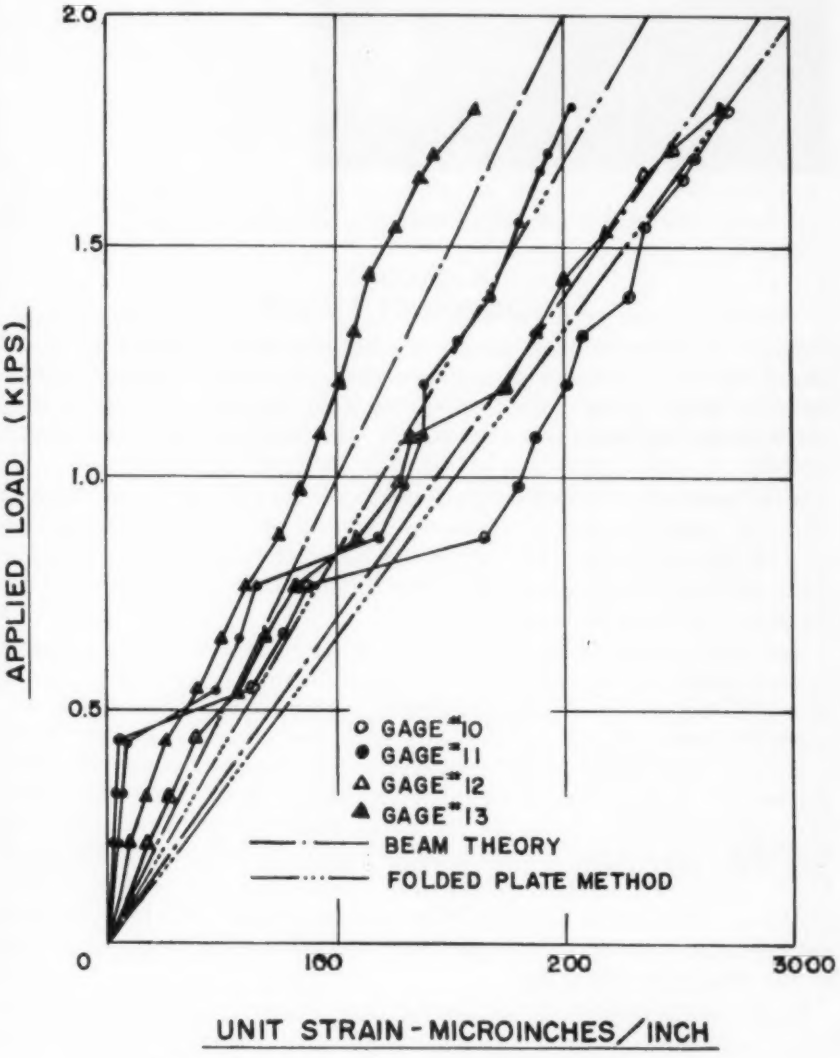

Fig. 5—Applied load versus main reinforcement strains

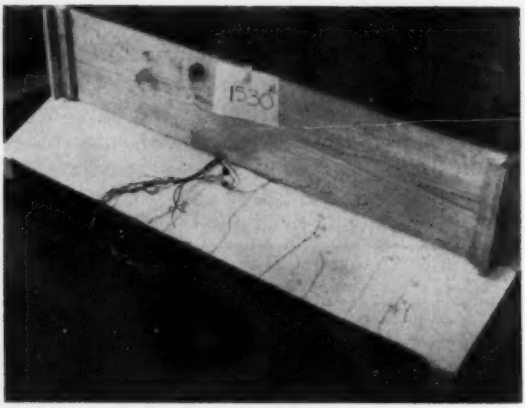

Fig. 6 — Crack pattern at 62.5 percent of ultimate load

A total load of 2457 lb was placed on the structure before collapse occurred.

DISCUSSION OF RESULTS

Strain readings recorded during the loading were compared to values calculated on the basis of the beam method, assuming a cracked section, and the folded plate method (Fig. 4 and 5). Modulus of elasticity of the concrete was assumed to be 1000 f_c' (1.5×10^6 psi) for these elastic studies.

Steel strains for the folded plate method are based on the assumption that the reinforcement provides the same moment about the neutral axis of the side plate as the tension wedge determined by plate action. Also, the load in each bar, and likewise strain, is proportional to the distance from this neutral axis.

The dissimilarity of strains during the early stages of loading from reinforcement in one side plate compared to the opposite plate is due to one side plate cracking at midspan during curing. This crack was generally perpendicular to the plate edge and extended nearly to the top plate. During the application of the fifth load the other side plate cracked to form a symmetrical section after which there was closer agreement between strains in the upper rods (Gages 11 and 13) and the lower rods (Gages 10 and 12) of the side plates. The abrupt changes in the plots of the observed strains indicate the occurrence of new cracking. The cracks usually developed and increased in pairs, one crack in each plate opposite each other and were generally perpendicular to the free edges (Fig. 6).

Strain readings taken at an applied load of 1908 lb indicated that the lower main reinforcement was near the proportional limit. Compression stress on the top fiber at this load is calculated to be 600 psi.

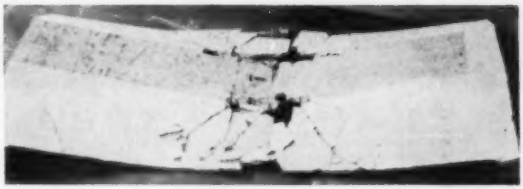

Fig. 7 — View of structure after failure of main reinforcement showing collapse damage

Vertical deflection at a load of 1197 lb, 48.6 percent of ultimate load, is 1/333 times the span. For a load of 1808 lb, 71.2 percent of ultimate load the deflection ratio is 1/163.

Failure occurred suddenly when both reinforcement rods in one side plate ruptured. Considerable damage was done to the center section of the structure during collapse, but the end sections remained intact (Fig. 7).

The ultimate moment developed was 21,500 in.-lb which compared well with a theoretical value of 21,900 in.-lb.

CONCLUSION

Based on one limited test the following conclusions may be stated.

The test of the three-plate folded structure indicated that the prediction of the ultimate moment based on the assumption of simple beam action was valid for this particular structure. The ultimate collapse load agreed with the theoretical load within 1.8 percent. This verification indicates that the Whitney method for predicting ultimate moment capacity is worthy of consideration in evaluating the strength of this type of structure.

The stress in the tensile steel behaved elastically until collapse occurred and agreed reasonably well with the beam and folded plate methods of analysis.

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CONCRETE BRIEFS

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Strength of Neat Cement Pastes Molded under Pressure

By F. J. LECZNAR* and R. M. BARNOFF†

THE COMPRESSIVE STRENGTH of a portland cement-water paste is influenced by several variables, the principal ones being water-cement ratio, methods of making and curing the specimens, and curing time.

It has been observed that the strength of concrete can be improved by consolidation of the mix. Vibration results in better consolidation and higher strength than is obtained without vibration. It has been supposed, as an extension of this observation, that further consolidation of the mix might greatly improve the compressive strength. Therefore, a series of specimens were "pre-compressed;" that is, they were squeezed at controlled pressures immediately after being placed in the forms. In the series of tests, compressive strengths of portland cement-water pastes were determined.

In pre-compressed cement-water pastes, the most important factors affecting compressive strength are intensity of molding pressure, time of curing, and water-cement ratio. To study the influence of these three variables, three sets of test specimens were made. The length of curing time was used as a variable in the first series, the intensity of molding pressure was varied in the second series, and the water-cement ratio was varied in the third series.

The specimens were prepared by filling a cylindrical mold 3.7 in. in diameter and 3.7 in. in height; then compressing the paste to the desired pressure, using a piston which fit closely within the cylindrical mold. The piston was then withdrawn, the remaining cavity of the mold filled, and the pressure was applied a second time. The resulting specimens varied slightly in total height—from about 3.5 to 3.6 in., averaging 3.54 in.

To demonstrate the mechanical effect of molding pressure, one specimen was made with no cement but with fine basalt grains (less than 0.2 mm). A water-basalt ratio of 0.06 was used, and a molding pressure of 10,080 psi was imposed. After 24 hr in air the specimen had lost 24 percent of its water but still possessed a compressive strength of 910 psi.

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TABLE 1 — EFFECT OF VARYING CURING TIME
Water-cement ratio—0.10; molding pressure—1420 psi; porosity—23.8 percent

Specimen No.	1-3	4-6	7-9	10-12	13-15
Curing time, days	3	7	14	21	28
Average compressive strength, psi	5320	7900	9940	11,790	13,490

TABLE 2 — EFFECT OF VARYING MOLDING PRESSURE
Water-cement ratio—0.10; curing time—7 days; porosity varies

Specimen No.	16-18	19-21	22-24	25-27	28-30	31-33
Molding pressure, psi	213	426	710	1420	2130	2840
Average compressive strength, psi	3410	5110	6110	7950	9440	10,080
Porosity, percent	33.1	29.8	27.3	23.8	20.2	18.9
Ratio of compressive strength to molding pressure	16.0	12.0	8.6	5.6	4.4	3.5

TABLE 3 — EFFECT OF VARYING WATER-CEMENT RATIO
Molding pressure—1420 psi; curing time—7 days; porosity varies

Specimen No.	34-36	37-39	40-42	43-45	46-48	49-51
Water-cement ratio	0.06	0.08	0.10	0.12	0.14	0.16
Average compressive strength, psi	7670	7680	7950	8380	8950	9080
Porosity, percent	31.5	27.9	23.8	19.5	15.4	12.0

One reference specimen was cast normally — that is, with no pre-compression. It had a water-cement ratio of 0.26 and after 7 days of water curing attained a compressive strength of 2130 psi.

Table 1 shows the compressive strength for Series A in which the water-cement ratio was held constant at 0.10 and the intensity of molding pressure was held constant at 1420 psi for several seconds. Three specimens from the series of 15 specimens were tested after water curing for each of the time periods: 3, 7, 14, 21, and 28 days. The porosity of the pre-compressed specimens was found to be 23.8 percent. Porosity was calculated from the weight of the molded specimen and a calculated solid weight of cement-water combination.

Table 2 shows the compressive strengths for Series B in which the water-cement ratio was again held constant at 0.10 and the curing time was held constant at 7 days. Three specimens from the series of 18 specimens were pre-compressed at each of the intensities: 213, 426, 710, 1420, 2130, and 2840 psi. The porosity of the pre-compressed specimens varied inversely with the intensity of molding pressure, and is shown in the tabulation. As a matter of interest, the ratio of compressive strength to molding pressure is also shown.

Table 3 shows the compressive strengths for Series C in which the intensity of molding pressure was held constant at 1420 psi and the curing time was held constant at 7 days. Three specimens from the series of 18 specimens were made with each of the water-cement ratios: 0.06, 0.08, 0.10, 0.12, 0.14, and 0.16.

The data in Table 1 illustrate that the compressive strength of pre-compressed cement pastes increases with

the length of curing time. This relationship can be expressed by the approximate empirical formula

$$R = a\sqrt{t}$$

where R = compressive strength in psi, t = curing time in days, and a is an empirical constant derived from the specimens tested.

An approximate formula showing the relationship between the compressive strength and the magnitude of molding pressure was derived from the data in Table 2:

$$R = b\sqrt{p}$$

where p = molding pressure in psi and b is also an empirical constant derived from the test specimens.

The influence of the water-cement ratio on the compressive strength of cement-water pastes is shown in Table 3. From Table 3, where the molding pressure and curing time were constant, we can observe that varying the water-cement ratio produced insignificant changes in the compressive strength of the specimens. When the water-cement ratio was varied from 0.06 to 0.16 the compressive strength of the specimens increased by approximately 18 percent.

Table 2 also illustrates the effect of molding pressure on compressive strength. From Table 2 the following empirical equation can be obtained:

$$R = c\sqrt{p}$$

where p = molding pressure in psi and c is a variable. Values of c may be obtained from Table 2.

In summary, the magnitude of molding pressure and length of curing time greatly affect the compressive strength of pre-compressed cement-water pastes, while varying the water-cement ratio produces practically an insignificant change in the compressive strength.

Locating Hold-Downs in Pretensioned Girder

By HERMAN TACHAU*

HOLD-DOWN POINTS for draped prestressing tendons in simple span pretensioned concrete girders can be located by mathematics. Two hold-downs (A) are used ordinarily, as shown in the diagramatic sketch, Fig. 1.

A parabola will be regarded as the ideal profile for the draped strands; but, in reality, the draped strands follow straight lines, as indicated in Fig. 1. The problem is to fit these lines as close as possible to the ideal curve.

Selection of the parabola as an ideal curve is based on the following considerations: At the time of initial prestress, when the girder is bent by its own weight only, the bending moment curve is a parabola. Likewise, for live load plus dead load the total design moments usually vary as a parabola. This has led many engineers to adopt the parabolic curve for the centroid of the prestressing steel; and this means, in turn, that the draped strands should follow a parabola.

The analysis which follows was worked out for use with bridge I-beams, but it is applicable to any prismatic pretensioned concrete beam.

In Fig. 2 the vertex of the parabola is at the origin which refers to the center of the span, and b is half the span length. The equation of the parabola is:

$$y = kx^2 \quad \text{.....(1)}$$

and the straight line, representing draped strands, is given by:

$$y = n(x - u) \quad \text{.....(2)}$$

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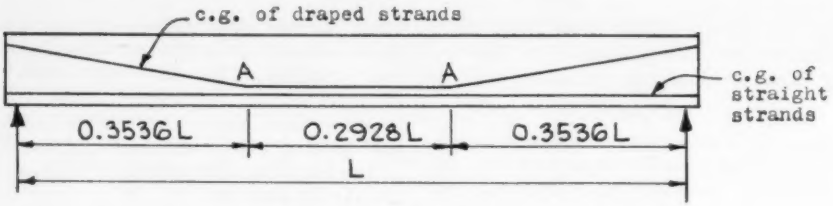


Fig. 1—Elevation of pretensioned concrete girder

All symbols are defined in Fig. 2, except constants k and n.

$$k = a/b^2 \quad \ldots\ldots (3)$$

and

$$n = \frac{a}{b-u} \quad \ldots\ldots (4)$$

Also,

$$r = ku^2 \quad \ldots\ldots (5)$$

and

$$s = n(v-u) - kv^2 \quad \ldots\ldots (6)$$

In order for the draped strands to be as close as possible to the parabola, equate $r = s$ and set $\partial s/\partial v = 0$ (to find the maximum value of s). Then

$$ku^2 = n(v-u) - kv^2 \quad \ldots\ldots (7)$$

and

$$n(1-0) - 2kv = 0$$

or

$$n = 2kv \quad \ldots\ldots (8)$$

Eq. (8) states that the slope of the parabola is parallel to the straight line at the point where s reaches a maximum. From Eq. (8)

$$v = \frac{n}{2k} \quad \ldots\ldots (8a)$$

Substitute Eq. (8a) in Eq. (7),

$$ku^2 - \frac{n^2}{2k} + nu + k\left(\frac{n}{2k}\right)^2 = 0$$

This can be written as:

$$\left(u + \frac{n}{2k}\right)^2 - \frac{n^2}{2k^2} = 0$$

Thus

$$u = -\frac{n}{2k} \pm \frac{n}{\sqrt{2}\,k}$$

The pertinent solution is:

$$u = \left(\frac{1}{\sqrt{2}} - \frac{1}{2}\right)\frac{n}{k}$$

$$= +\ 0.2071\,\frac{n}{k} \quad \ldots\ldots (9)$$

Substituting Eq. (3) and (4) in Eq. (9), one obtains:

$$u = 0.2071\,\frac{a}{b-u}\left(\frac{b^2}{a}\right)$$

Multiply both sides by $(b-u)$ and transpose terms,

$$u^2 - ub + 0.2071\,b^2 = 0$$

and solve for:

$$u = \frac{b \pm \sqrt{b^2 - 4(0.2071)\,b^2}}{2}$$

$$u = \frac{b}{2}\,(1 \pm \sqrt{0.1716})$$

$$= \begin{cases} +\ 0.7071b \quad \ldots\ldots (10a) \\ +\ 0.2929b \quad \ldots\ldots (10b) \end{cases}$$

By inspection Solution (10b) is applicable here.

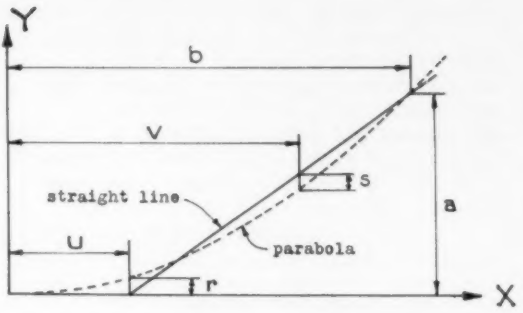
Further, from Eq. (3), (4), (8a), and (10b)

$$v = \frac{a}{b-u}\left(\frac{b^2}{2a}\right) = \frac{b^2}{2\,(b-0.2929b)}$$

$$= \frac{b}{2\,(0.7071)}$$

$$v = 0.7071b \quad \ldots\ldots (11)$$

Fig. 2—Geometric set-up for finding hold-down point *u*



and from Eq. (5), (6), (10b), and (11)

$$r = s = 0.0858a \quad \text{.....(12)}$$

According to Eq. (10b) u is independent of a. In other words, the location of hold-downs is not related to the rise of the curve. This property seems to be peculiar to the parabola.

Results identical to Fig. 1 were obtained originally through successive approximations by William J. Cahalan, bridge designer with the New Mexico State Highway Department. Mr. Cahalan developed his analysis while designing a series of precast pretensioned I-beam bridge spans. The writer checked the design and participated in preparing standard plans. The bridge office was then headed by W. E. Strohm, formerly bridge engineer. This work was carried out under the guidance of Charles E. Reed, now the bridge engineer, New Mexico State Highway Department, who contributed valuable ideas and checked the mathematics.

Square Column with Double Eccentricities Solved by Numerical Method

By CHARLES CHO-LIM ANG*

IN ACCORDANCE with current codes and common practice, a reinforced concrete column which is subject to combined bending in two directions and axial load may be designed based on the "cracked section" theory if the eccentricity is large. Although a few papers have been published dealing with this subject, the tedious mathematical operation has never been overcome, except to be challenged by the recent electronic computer analysis. A numerical method in tabular form is presented herewith as a companion tool in solving problems of this nature.

Notation

P = axial concentrated load
$M_{1\text{-}1}$, $M_{2\text{-}2}$ = moment about axes 1-1 and 2-2, respectively
C_1, C_2 = distance to extreme fiber from axes 1-1 and 2-2, respectively
A = transformed areas of steel and concrete (uncracked section)
$I_{1\text{-}1}$, $I_{2\text{-}2}$ = moment of inertia of uncracked section about axes 1-1 and 2-2, respectively

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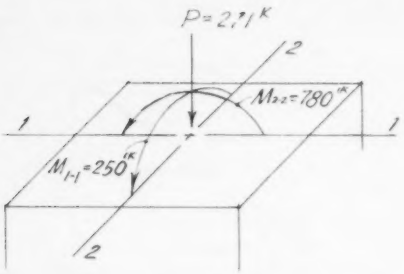

Fig. 1—Loaded section

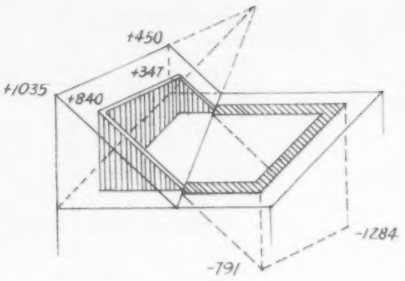

Fig. 2—Stress volume

M_{x-x}, M_{y-y} = moment about axes X-X and Y-Y, respectively

e_y, e_x = eccentricity from axes Y-Y and X-X, respectively

y, x = distance to centroid of stress volume (see Fig. 2) from axes X-X and Y-Y, respectively; coordinates of stress points

u, v = distance measured along X-X and Y-Y axes, respectively, of the base of prism of stresses (see Fig. 3)

A_s = steel area

a, b, c, d, e, f = transformed steel band representing A_s

p = ratio of steel area to gross concrete area

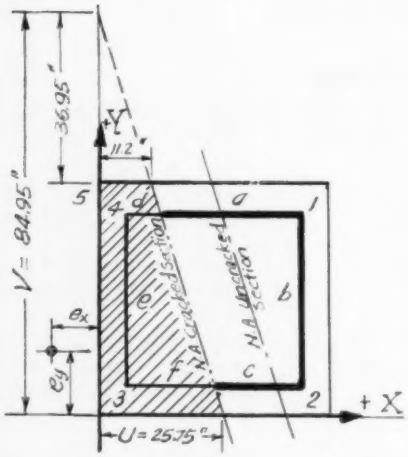

Fig. 3—Design section

f_c = maximum allowable compressive stress in members subjected to combined axial and bending stresses

f_c = concrete stress

f_s = steel stress

Description

To illustrate this method, an actual problem has been analyzed and solved numerically in a tabular form prepared to facilitate the operation (Table 1). The actual loading of the column is shown in Fig. 1 with the directions and magnitude assigned. The approach to the problem is outlined in the following general manner:

1. With some degree of experience and good judgment, 20–#11 bars are assumed to reinforce a 4 x 4-ft column. Treating the columns as an uncracked section, the stresses can be computed by using the familiar formula for eccentrically loaded columns:

$$\frac{P}{A} \pm \frac{M_{1-1}\,C_1}{I_{1-1}} \pm \frac{M_{2-2}\,C_2}{I_{2-2}}$$

where the properties of the transformed section are employed with a modular ratio $n = E_s/E_c = 10$. As a result, the neutral axis can be located on the plane of the column section by direct proportioning of the corner stresses either numerically or graphically.

2. Assuming the position of the neutral axis of the cracked section to be parallel to that of the uncracked section, the stress at any point of the

plane can be found by Bernoulli's plane theorem:

$$f_s = f_s \left(1 - \frac{x}{u} - \frac{y}{v} \right).$$

For the trial section, f_s value is assumed to be based on an approximate calculation using the formulas specified in the 1957 AASHO "Standard Specifications for Highway Bridges."

3. Trial runs are conducted by iteration with various positions of the neutral axis of the uncracked section closely parallel to that of the cracked section until the resisting P, M_{x-x}, and M_{y-y} balance out that of the actual

TABLE 1 — DESIGN PROBLEM (FIG. 3)

$P = 271$ kips
$M_{1-1} = 250$ ft-kips
$M_{2-2} = 780$ ft-kips
$e_y = +12.9$ in.
$e_x = -10.6$ in.

I (gross transformed) $= 516{,}500$ in.^{4}

$$f_c = \frac{271}{2580} \pm \frac{250 \times 24}{516{,}500} \pm \frac{780 \times 24}{516{,}500}$$
$$= 0.105 \pm 0.116 \pm 0.362$$

Assume $f_c = 1035$ psi*

Point (Fig. 3)	x	y	$f_c = f_c \left(1 - \frac{x}{u} - \frac{y}{v} \right)$, psi
1	44.29	44.29	$1035 \left(1 - \frac{44.29}{25.75} - \frac{44.29}{84.95} \right) = -1284$
2	44.29	3.71	$1035 \left(1 - \frac{44.29}{25.75} - \frac{3.71}{84.95} \right) = -791$
3	3.71	3.71	$1035 \left(1 - \frac{3.71}{25.75} - \frac{3.71}{84.95} \right) = +840$
4	3.71	44.29	$1035 \left(1 - \frac{3.71}{25.75} - \frac{44.29}{84.95} \right) = +347$
5	0	48.00	$1035 \left(1 - \frac{0}{0} - \frac{48.00}{84.95} \right) = +450$

Note:
+ — compressive stress
− — tensile stress

*AASHO specification:

$$\frac{Ke}{t} = \frac{5.75 \times 3.02 \times 12}{48} = 4.34 \text{ in.}$$

Using 20—#11 bars, $A_s = 31.2$ sq in. $p = 31.2/2304 = 1.35$ percent

Column: 4 x 4 ft A_s per band $= 5 \times 1.56 = 7.80$ sq in.

		Steel band, in.		Stress volume	P, lb	y	x	M_{x-x}, in.-lb	M_{y-y}, in.-lb
		Length	Width						
Steel	a	31.93	$\frac{nA_s/40.59}{10 \times 7.80} = \frac{1.92}{40.59}$	$-\frac{1284}{2} \times 31.98 \times 1.92$	− 39,419	44.29	33.63	−1,745,859	−1,325,654
	b	40.59		$-\frac{791}{2} \times 40.59 \times 1.92$	− 61,643	24.00	44.29	−1,479,423	−2,730,152
				$-\frac{493}{2} \times 40.59 \times 1.92$	− 19,207	30.76	44.29	− 590,816	− 850,690
	c	19.67		$-\frac{791}{2} \times 19.67 \times 1.92$	− 14,934	3.71	37.73	− 55,405	− 563,463
	d	8.61	$\frac{(n-1)A_s/40.59}{9 \times 7.80} = \frac{1.73}{40.59}$	$+\frac{347}{2} \times 8.61 \times 1.73$	+ 2,599	44.29	6.58	+ 115,111	+ 17,102
	e	40.59		$+\frac{347}{2} \times 40.59 \times 1.73$	+ 24,366	24.00	3.71	+ 584,792	+ 90,399
				$+\frac{493}{2} \times 40.59 \times 1.73$	+ 17,408	17.24	3.71	+ 300,111	+ 64,583
	f	20.92		$+\frac{840}{2} \times 20.92 \times 1.73$	+ 15,288	3.71	10.68	+ 56,718	+ 163,276
Concrete				$+\frac{1035}{3} \times \frac{25.75 \times 84.95}{2}$	+377,382	21.24	6.46	+8,015,587	+2,437,886
				$-\frac{450}{3} \times \frac{11.2 \times 36.95}{2}$	− 31,038	57.24	2.80	−1,776,615	− 86,906
				$\Sigma =$	+270,802			+3,424,201	−2,783,629

$f_c = 1035$ psi $f_s = -12{,}840$ psi (tension) $f_s = +8400$ psi (compression) $e_y = +12.64$ in. $e_x = -10.28$ in.

loading. The final result may be obtained in two to three trials.

The author wishes to emphasize that the solution presented above is by no means restricted to a square column. Each individual designer may exercise his own ingenuity to apply the foregoing method to as many analogous problems as conceivable.

Ultimate Strength Design of Beams with the Aid of Tables

By A. ZASLAVSKY*

IN ULTIMATE STRENGTH design of beam sections (ACI 318-56) advantageous use may be made of tables, which are often more convenient to use than charts.[1-4]

The relevant formulas (see Fig. 1) are:

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{1.176 p d f_y}{f_c'} \quad \text{(1)}$$

$$M_u = Ti = bd^2 p f_y (1 - 0.59\, p f_y / f_c') = K\, bd^2 \quad \text{(2)}$$

where:

$$p = \frac{A_s}{bd} \quad \text{(3)}$$

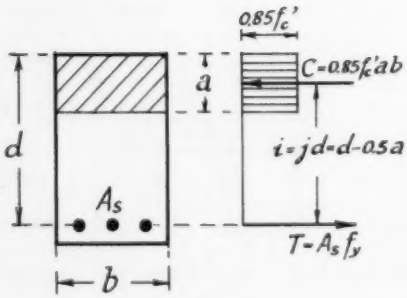

Fig. 1

Assuming p and b, d may be found from Eq. (2) for given ultimate bending moment M_u, f_y, and f_c'. After d is obtained, A_s is calculated from Eq. (3):

$$A_s = pbd \quad \text{(3a)}$$

or alternatively directly from:

$$A_s = \frac{M_u}{if_y} = \frac{M_u}{jdf_y} \quad \text{(2a)}$$

Eq. (2) may be transformed into:

$$d = \sqrt{\frac{1}{pf_y (1 - 0.59\, f_y/f_c')}} \sqrt{\frac{M_u}{b}} = \rho \sqrt{\frac{M_u}{b}} \quad \text{(4)}$$

The three formulas for the use of the tables are therefore:

$$d = \rho \sqrt{\frac{M_u}{b}}$$

$$M_u = K\, bd^2$$

$$A_s = pbd$$

In Table 1 the coefficients j, ρ, and K† are given for steel of $f_y = 50{,}000$ psi and concretes of $f_c' = 3000$ psi, $f_c' = 4000$ psi, and $f_c' = 5000$ psi. The reinforcement ratio p is given in percent and M_u is to be expressed in ft-lb; d and b in in.

According to the Code, p is limited to:

$$p_{max} = 0.40\, f_c'/f_y \quad \text{(5)}$$

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†$K = 1/\rho^2 = jpf_y/12$.

TABLE 1 — COEFFICIENTS FOR ULTIMATE STRENGTH DESIGN OF RECTANGULAR BEAM SECTIONS (Steel grade: $f_y = 50{,}000$ psi)

p, percent	$f_c' = 3000$ psi			$f_c' = 4000$ psi			$f_c' = 5000$ psi			p, percent
	j	ρ	K	j	ρ	K	j	ρ	K	
0.04	0.996	0.776	1.66	0.997	0.776	1.66	0.998	0.776	1.66	0.04
0.06	0.994	0.634	2.49	0.996	0.634	2.49	0.996	0.634	2.49	0.06
0.08	0.992	0.550	3.31	0.994	0.549	3.31	0.995	0.549	3.32	0.08
0.10	0.990	0.492	4.13	0.993	0.492	4.14	0.994	0.491	4.14	0.10
0.12	0.988	0.450	4.94	0.991	0.449	4.96	0.993	0.449	4.96	0.12
0.14	0.986	0.417	5.75	0.990	0.416	5.77	0.992	0.416	5.79	0.14
0.16	0.984	0.390	6.56	0.988	0.390	6.59	0.991	0.389	6.60	0.16
0.18	0.982	0.368	7.37	0.987	0.368	7.40	0.989	0.367	7.42	0.18
0.20	0.980	0.350	8.17	0.985	0.349	8.21	0.988	0.348	8.24	0.20
0.25	0.975	0.314	10.16	0.982	0.313	10.22	0.985	0.312	10.26	0.25
0.30	0.971	0.287	12.13	0.978	0.286	12.22	0.982	0.285	12.28	0.30
0.35	0.966	0.266	14.08	0.974	0.265	14.21	0.979	0.266	14.14	0.35
0.40	0.961	0.250	16.01	0.971	0.249	16.18	0.976	0.248	16.27	0.40
0.45	0.956	0.236	17.92	0.967	0.235	18.13	0.973	0.234	18.25	0.45
0.50	0.951	0.225	19.81	0.963	0.223	20.07	0.971	0.222	20.22	0.50
0.55	0.946	0.215	21.68	0.959	0.213	21.99	0.968	0.212	22.17	0.55
0.60	0.941	0.206	23.53	0.956	0.205	23.89	0.965	0.204	24.12	0.60
0.65	0.936	0.199	25.35	0.952	0.197	25.79	0.962	0.196	26.05	0.65
0.70	0.931	0.192	27.16	0.948	0.190	27.66	0.959	0.189	27.96	0.70
0.75	0.926	0.186	28.95	0.945	0.184	29.52	0.956	0.183	29.87	0.75
0.80	0.921	0.180	30.71	0.941	0.179	31.37	0.953	0.177	31.76	0.80
0.85	0.916	0.176	32.46	0.937	0.174	33.20	0.950	0.172	33.64	0.85
0.90	0.912	0.171	34.18	0.934	0.169	35.01	0.947	0.168	35.51	0.90
0.95	0.907	0.167	35.89	0.930	0.165	36.81	0.944	0.164	37.37	0.95
1.00	0.902	0.163	37.57	0.926	0.161	38.59	0.941	0.160	39.21	1.00
1.08	0.894	0.158	40.22	0.920	0.155	41.42	0.936	0.154	42.13	1.08
1.10	0.892	0.156	40.88	0.919	0.154	42.11	0.935	0.153	42.86	1.10
1.20	0.882	0.151	44.10	0.912	0.148	45.58	0.929	0.147	46.46	1.20
1.30	0.872	0.145	47.24	0.904	0.143	48.97	0.923	0.141	50.01	1.30
1.40	0.862	0.141	50.30	0.897	0.138	52.31	0.917	0.138	53.52	1.40
1.44	0.858	0.139	51.50	0.894	0.137	53.63	0.915	0.135	54.90	1.44
1.50	0.853	0.137	53.28	0.889	0.134	55.59	0.912	0.132	56.97	1.50
1.60	0.843	0.133	56.18	0.882	0.130	58.80	0.906	0.129	60.37	1.60
1.70	0.833	0.130	58.99	0.875	0.127	61.95	0.900	0.125	63.73	1.70
1.80	0.823	0.127	61.73	0.867	0.124	65.04	0.894	0.122	67.04	1.80
1.90	0.813	0.125	64.38	0.860	0.121	68.07	0.888	0.119	70.29	1.90
2.00	0.803	0.122	66.94	0.853	0.119	71.04	0.882	0.117	73.50	2.00
2.10	0.794	0.120	69.43	0.845	0.116	73.95	0.876	0.114	76.66	2.10
2.20	0.784	0.118	71.84	0.838	0.114	76.79	0.870	0.112	79.77	2.20
2.30	0.774	0.116	74.16	0.830	0.112	79.58	0.864	0.110	82.83	2.30
2.40	0.764	0.114	76.00	0.823	0.110	82.30	0.858	0.108	85.84	2.40
2.50				0.816	0.108	84.96	0.853	0.106	88.80	2.50
2.60				0.808	0.107	87.56	0.847	0.104	91.72	2.60
2.70				0.801	0.105	90.10	0.841	0.103	94.58	2.70
2.80				0.794	0.104	92.58	0.835	0.101	97.39	2.80
2.90				0.786	0.103	94.99	0.829	0.100	100.16	2.90
3.00				0.779	0.101	97.34	0.823	0.099	102.88	3.00
3.10				0.771	0.100	99.64	0.817	0.097	105.54	3.10
3.20				0.764	0.099	101.87	0.811	0.096	108.16	3.20
3.30							0.805	0.095	110.73	3.30
3.40							0.799	0.094	113.25	3.40
3.50							0.794	0.093	115.72	3.50
3.60							0.788	0.092	118.14	3.60
3.70							0.782	0.091	120.51	3.70
3.80							0.776	0.090	122.84	3.80
3.90							0.770	0.089	125.10	3.90
4.00							0.764	0.089	127.33	4.00

Note: Coefficients lying below dividing lines refer to values of p where deflections must be investigated.

and the value p' above which deflections must be investigated is limited to:

$$p' = 0.18\, f_c'/f_y = 0.45\, p_{max} \quad \ldots\ldots (6)$$

In the table the coefficients belonging to p' are underlined.

The table confirms that concrete strength f_c' has little influence on the ultimate bending capacity of the section (coefficients K and ρ), but, of course, it affects the values of p_{max} and p'.

The slow change of j (with changing p) is also indicated. In calculating A_s, small changes of d may, therefore, be accounted for proportionally (see Example 3).

Example 1 — Calculate the ultimate bending capacity M_u for given section: $b = 10$ in., $d = 20$ in., $A_s = 1.8$ sq in., $f_y = 50{,}000$ psi, $f_c' = 4000$ psi.

$$p = \frac{A_s \times 100}{bd} = \frac{1.8 \times 100}{10 \times 20}$$

$$= 0.9 \text{ percent}$$

From Table 1, $K = 35.01$, hence:

$$M_u = K\, bd^2 = 35.01 \times 10 \times 20^2$$

$$= 140{,}040 \text{ ft-lb}$$

Example 2 — Calculate reinforcement for slab of depth $d = 6$ in. and ultimate bending moment $M_u = 120{,}000$ in.-lb (per 12-in. strip). Materials: $f_y = 50{,}000$ psi, $f_c' = 3000$ psi. Expressing M_u in ft-lb we calculate:

$$K = \frac{M_u}{bd^2} = \frac{120{,}000/12}{12 \times 6^2}$$

$$= \frac{10{,}000}{12 \times 6^2} = 23.15$$

From Table 1 we find (after interpolation) the corresponding reinforcement ratio to be $p \cong 0.59$ percent and:

$$A_s = pbd = 0.0059 \times 12 \times 6$$

$$= 0.425 \text{ sq in.}$$

or alternatively:

$$\rho = \frac{d}{\sqrt{M/b}} = \frac{6}{\sqrt{10{,}000/12}} \cong 0.208$$

From the table we find correspondingly $p \cong 0.59$ percent, etc. Since differences are small, interpolation between ρ values is somewhat easier than between K values.

Example 3 — Design beam section for ultimate bending moment of $M_u = 150{,}000$ ft-lb. Materials as before: $f_y = 50{,}000$ psi, $f_c' = 3000$ psi.

Assuming $b = 10$ in. and choosing:

$$p = 1.1 \text{ percent} \cong p'$$

we find from the table the corresponding depth coefficient $\rho = 0.156$:

$$d = \rho \sqrt{M/b} = 0.156 \sqrt{150{,}000/10}$$

$$= 19.1 \text{ in., say } 20 \text{ in.}$$

Since d was increased from 19.1 in. to 20 in. we now reduce the reinforcement ratio accordingly:

$$p = 1.1 \frac{19.1}{20} \cong 1.05 \text{ percent}$$

$$A_s = pbd = 0.0105 \times 10 \times 20 = 2.1 \text{ sq in.}$$

* * * *

Tables similar to Table 1 have been prepared for $f_y = 40{,}000$ psi steel and $f_y = 60{,}000$ steel. These are available from ACI headquarters at cost of reproduction—50 cents per set.

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3. Ferguson, Phil M., "Simplification of Design by Ultimate Strength Procedures," *Transactions*, ASCE, V. 123, 1958, pp. 602-628.

4. Amstad, R. J., "Ultimate Strength Design of Rectangular Cross Sections Under Bending with Tensile Steel Only," ACI JOURNAL, V. 31, No. 9, Mar. 1960 (*Proceedings* V. 56), pp. 884-886.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Design and construction of the Forster-Tuncurry Bridge

R. MILTON JOHNSON and G. P. COOK, *Journal*, Institution of Engineers, Australia (Sydney), V. 32, No. 7-8, July-Aug. 1960, pp. 157-164

Deals with all phases in the construction of the Forster-Tuncurry Bridge. The bridge is of particular interest for it is the longest short-span prestressed concrete bridge yet constructed in Australia.

High-strength reinforcing steels for concrete bridges

EIVIND HOGNESTAD and GEORGE WINTER, *Proceedings*, Highway Research Board, V. 39, 1960, pp. 103-119

AUTHORS' SUMMARY

The use of high-strength deformed bars without prestressing in European concrete structures is reviewed, with particular emphasis on bridge construction. American use of high-strength reinforcement has so far included only buildings. However, ASTM specifications for 60,000- and 75,000-psi yield point reinforcement have recently been issued, and an increasing future use in bridge structures may be expected.

To supplement other research information, chiefly from abroad, on rectangular beams reinforced with cold-twisted steel, 42 T-beams reinforced with metallurgically produced high-strength steels have been tested at Cornell University. To simulate continuity, these were single-span beams with overhangs. Yield points ranged from 84,000 to 103,000 psi. The tests showed that the strengths and deflections of these beams conformed to accepted theories with about the same agreement as for members with conventional strength reinforcement. Crack width at design loads can be held within acceptable limits if adequate attention is given to detailing.

Tancarville Bridge (Le Pont de Tancarville)

MARCEL HUET, *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. 113, No. 5, May 1960, pp. 23-42

Reviewed by RICHARD CEBULA

The Tancarville Bridge over the Seine is located 28 km from Le Havre, was opened on July 2, 1959, and is the longest bridge in Europe. The total length is 1420 m. The 608-m central span of this toll bridge is suspended 50 m above the water and must resist strong winds. The 123-m height of the towers makes them the highest towers ever built of reinforced concrete.

Parramatta River Bridge, New South Wales

The Engineer (London), V. 209, No. 5438, Apr. 15, 1960, pp. 640-641

Reviewed by ARON L. MIRSKY

New bridge, over an arm of Sydney Harbor, will be a reinforced concrete arch of 1000-ft span. There will be four ribs, each of precast box section 20 ft wide and varying in depth from 12 ft at the crown to 23 ft 6 in. at the springing; the ribs will be joined by in-situ concrete, and made to act

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monolithically by prestressed diaphragms. To ensure the correct final arch profile, Freyssinet jacks are provided at the two points of contraflexure. Centering will be of steel, designed for multiple use.

Arch falsework moves on rails

Contractors and Engineers, V. 57, No. 8, Aug. 1960, pp. 66-69

Emphasizes construction difficulties of the $540,000 double-rib reinforced concrete arch bridge over Capilano Canyon, North Vancouver, B. C., Canada.

The arch falsework was limited to four piers and subject to severe flood conditions. Each of the two arch ribs has a span of 350 ft and a rise of 57 ft from abutment to center. Hollow arch ribs were used to decrease dead weight.

Preloading bridge abutments

Construction World (Vancouver), V. 15, No. 13, Oct. 1960, pp. 34-37

HIGHWAY RESEARCH ABSTRACTS V. 30, No. 10, Nov. 1960

A river bank of boulders and gravel running more than 200 ft deep has given rise to a new approach in bridge building in British Columbia. It was deemed necessary to determine in advance whether movement would occur in the abutments under stress of the steel in place.

To accomplish such a crossing only an arch layout was feasible, with regard to the approach conditions and the long span needed to cross the canyon. And support of an arch bridge depends on the fixity of the arch abutments.

Because no solid rock was found for the abutments, they not only had to be designed for proper distribution of the load, but some method had to be found to predetermine the reaction of the abutments to the future dead load of the bridge. If movements resulted, they could be measured and recorded, and suitable adjustments could be made prior to and during steel erection.

The only solution was to pre-load the abutments with a simulated load equal to the load and its distribution of the completed bridge. The procedure of preloading the abutments with the future dead load of the bridge—4000 tons vertical and 1800 tons horizontal—was simple in theory. But its application involved many months of complex preparation and construction.

The main work phases were:

1. Drilling twelve 6-in. diameter holes through each abutment and through the overburden into solid rock, down as deep as 310 ft.
2. Making 4 ½-in. diameter cables of 246,000 psi ultimate strength wire strands, one for each hole.
3. Threading the cables down the holes and grouting them into solid rock.
4. Providing a jacking arrangement for each cable.
5. Preparing a tunnel behind each abutment and placing nine 3 ft thick jacking pads against the bank for two rows of jacks supplying the horizontal load.

The abutments, of massive, heavily reinforced concrete (3600 cu yd each) had already been placed.

Results of the complex test bore out the engineers' expectations: there was no appreciable movement, and construction of the bridge could go ahead as designed.

Construction

Observation, television, and radio tower in Dortmund (Der Aussichts -und Fernmeldeturm in Dortmund)

L. FAERBER, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 5, May 1960, pp. 97-102

Reviewed by RUDOLPH SZILARD

The design of the 720 ft tall reinforced concrete observation, television, and radio tower is described. The diameter of the cylindrical tower is 38 ft at the bottom and 18 ft at the top, thus the ratio of the height to the cross-sectional area is 19:1. The thickness of the reinforced concrete wall varies from 2 ft to 8 in. For the design the dynamic wind load was the most criti-

cal. Excellent soil condition minimized the foundation problem.

Construction of the observation, television and radio tower in Dortmund (Die Ausfuhrung des Dortnumder Aussichts—und Fernmeldeturms)

G. GURLITT, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 5, May 1960, pp. 102-106
Reviewed by RUDOLPH SZILARD

A detailed description of the construction of the Dortmund TV tower. Materials, sliding formwork, and foundation methods are discussed in detail.

Compressed air shield tunneling under the Rhine (Druckluft-Schildvortrieb unter dem Rhein)

FRIEDRICH HESS and HELMUT DOMZIG, *VDI Zeitschrift* (Düsseldorf), V. 102, No. 20, July 11, 1960, pp. 848-852
Reviewed by ARON L. MIRSKY

Compressed air tunneling using a shield was selected for construction of inverted siphon sewer under the Rhine at Düsseldorf, rather than the usual steel pipe laid in an open dredged channel. Siphon is approximately 905 m long, and maximum daily advance was 11.4 m. Prefabricated reinforced concrete ring segments were used for approximately 300 m of the tunnel, to check the technical and economic advantages of this type of construction as compared to the usual cast iron plates. [See also *Engineering News-Record*, June 23, 1960, pp. 110-111.]

Concrete-framed building to resist mining subsidence

The Engineer (London), V. 209, No. 5433, Mar. 11, 1960, p. 439
Reviewed by ARON L. MIRSKY

Brief illustrated description of two structures of the college of further education at Ystrad Mynach, near Cardiff, which include provision for releveling by jacking should subsidence occur. The four-story classroom-laboratory-office building, 136 ft x 60 ft 8 in., consists of two structures, 60 and 76 ft, respectively, separated by a 3 in. structural break; the two-story workshop block is 103 ft x 60 ft 8 in. Both structures utilize a central core ("spine") consisting of two rows of reinforced concrete columns which support at each floor level a precast prestressed concrete truss cantilevering out on either side of the core; the columns rest, in the case of the four-story structure, on a prestressed concrete foundation beam of box section which in turn is supported by mass concrete foundations, while the columns for the two-story workshop are supported by concrete foundation trusses resting on mass concrete foundations.

Exhibition hall of the National Center of Industries and Technology (in French)

N. ESQUILLAN, ET. AL., *Travaux* (Paris), V. 43, No. 296, June 1959, pp. 341-392
Reviewed by ETIENNE D. ROLIN

An engineering giant for times to come; this building is covered by a floor permitting access for installation of triangular domed roof supported at three points, forming an equilateral triangle with sides 715 ft long. The roof consists of two superimposed 2⅝ in thick concrete shells, 6 ft apart, cast in place with precast diaphragms to effectuate a series of box sections, radiating from the three supports.

The floor area, 1,120,000 sq ft, includes four storied constructions on each side of the triangle independent of the main dome. The main floor under the dome is supported 39 ft above the ground by concrete columns spaced on a 59 ft triangular grid. Each 59 ft triangular bay consists of 18 modular precast concrete triangular slabs in two layers 6 ft- 6 in. apart. Girders between columns are cast in place and prestressed with straight cables up to 470

Construction Techniques

Floor and roof deck systems

Building Construction, V. 30, No. 5, Nov. 1960, pp. 17-21, 28-29, 31, 41

In a number of short articles some of the floor and roof deck systems are pictured and described as used in various projects. (1) "Deck Forming Sys-

tem Allows 40 Percent Saving" describes prefabricated rolling scaffold shore where the deck casting area was quickly formed. Grids permitted assembly line production. (2) "System 'Clears Deck' for Fast Pan Forming" describes mass production methods in pan form layout. (3) "How to Handle Steel Domes for Waffle Floor" gives reasons for the selection of 19 in. square domes for a floor system that put the joists on a 4 ft planning module. (4) "Tips on Placing Cellular Steel Decking" pictures details of installation. (5) "New Slab Technique Forms Unique Deck" discusses a combination of structural concrete and perlite insulating concrete placed over the same steel dome forms.

Form technology considerations: What constitutes the "revolution" in form construction, evoked by the form panel? (Schalungstechnische Uberlegungen: Worin besteht die "Revolutionierung" im Schalungsbau, hervorgerufen durch die Schalplatte?)

FRITZ LETTNER, *Die Bauzeitung* (Stuttgart), V. 64, No. 9, Sept. 1959, pp. 405-406, 408, 410, 412

Reviewed by ARON L. MIRSKY

General, not-too-technical study of prefabricated form panels, with the emphasis on costs: materials, labor, incidental costs, nails, transportation, storage, etc.). Other factors discussed include the cutting and fitting of standard panels to suit beam sizes and spacings, and the various surfaces (good and bad) obtained with the panels.

Extension of the Mannesmann ore harbor by reinforced concrete quay wall, anchored by prestressing, with caisson foundation (Erweiterung des Erzhafens von Mannesmann durch eine vorgespannt verankerte Kaimauer in Stahlbeton mit Caisson-Gruendung)

HANNS LEHMANN, *Der Bauingenieur* (Berlin), V. 34, No. 9, Sept. 1959, pp. 363-368

Reviewed by ARON L. MIRSKY

Ore-unloading facilities of the Mannesmann steel mills, off the Rhine at Duisburg, required expansion in 1957-58. Many solutions were proposed for the quay wall (eight are sketched on p. 364); the selection was a thin reinforced concrete wall, founded on caissons and anchored to deadmen; the anchorage cables were prestressed and then grouted inside the protective sheaths.

Dams

Design of arch dams by trial load method of analysis

MERLIN D. COPEN, *Proceedings*, ASCE, V. 86, PO 4, Aug. 1960, pp. 13-29

AUTHOR'S SUMMARY

A brief résumé of the basic principles involved in the trial-load method of stress analysis is presented. The various types of analyses, and their advantages and limitations are examined. The application of trial-load methods to design problems and practical examples resulting from these procedures are illustrated and described in detail. It is also shown that the use of the trial-load method of stress analysis makes possible economy not only in the design of arch dams, but also in the time and expense required to produce such designs.

Concrete plant at Klang Gates Dam in Malaya

J. CONACHER, *The Engineer* (London), V. 209, No. 5434, Mar. 18, 1960, pp. 472-474

Reviewed by ARON L. MIRSKY

Design and construction of refrigeration plant used to precool the mass concrete for the Klang Gates Dam, Kuala Lumpur, Malaya. There were four separate plants, devoted to air cooling the aggregates, cooling the mixing water, manufacturing ice, and storing ice, respectively. Sample design calculations are given.

Digital computers for trial-load analysis of arch dams

L. R. SCRIVNER, *Proceedings*, ASCE, V. 86, PO 4, Aug. 1960, pp. 1-12

AUTHOR'S SUMMARY

Stress analysis of concrete dams is another of the ever-growing list of engineering problems that can be ex-

pedited by the use of electronic computers. The use of the "trial-load method" has been restricted due to the time-consuming, laborious computations that were required. The programming of many of these operations for an electronic computer results in a saving of time and costs, making this method of analysis more effective in the design of arch dams.

Arch dam analysis with an electric analog computer

RICHARD H. MACNEAL, *Proceedings*, ASCE, V. 86, EM 4, Aug. 1960, pp. 127-151
AUTHOR'S SUMMARY

An electrical analogy that simulates the elastic deformations of arch dams is described. This analogy consists of resistors, transformers, and current generators. The results of an analog computer analysis of the Stevenson Creek Dam are compared with field measurements. The results of a design study of the Blue Ridge Dam are summarized.

Design

Investigation of the skewed slab with automatic computers (Untersuchung der schiefen Platte mit Benutzung des Rechenautomaten)

MASAO NARUOKA, *Der Bauingenieur* (Berlin), V. 34, No. 10, Oct. 1959, pp. 401-406
Reviewed by ARON L. MIRSKY

Presented here are tables of influence coefficients for the deflections of and moments in bridge slabs of parallelogram shape, simply supported on all four sides (by the stringers and the end diaphragms), for length-width ratios of 2.0 (skew angle 30 and 45 deg) and 3.0 (skew angle 60 deg). Finite-difference methods as developed by Brigatti, Vogt, and Jensen at the University of Illinois, with a rectangular network for the 45 deg slab and triangular networks for the 30 and 60 deg slabs, were used. Calculations were done on a Facom-128 (Fuji Relay Automatic Computer). The application of the coefficients is shown by an example.

Design of curved reinforced concrete beams (Berechnung des gerkruemmten Stahlbetonbalkens)

B. TOPALOFF, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 5, May 1960, pp. 113-117
Reviewed by RUDOLPH SZILARD

Starting out from the well known theory of homogeneous curved beams, a method is derived for unhomogeneous reinforced concrete beams in the elastic region. Finally the plastic method is mentioned briefly. Diagrams facilitate the numerical computation.

Cylindrical shells: Computation by means of polynoms (Kreiszylinderschalen, Berechnung mit Polynomen)

W. FUCHSSTEINER, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 5, May 1960, pp. 106-113
Reviewed by RUDOLPH SZILARD

A simplified theory of cylindrical shells is treated. The moments in the longitudinal direction and the torsional moments are neglected in setting up the differential equations of equilibrium. The solution utilizes the expansion into infinite series. The solution is based on an other assumption, i.e., the longitudinal axial forces vary like the cosine.

New auxiliary tables for analysis of stress problems of second-order theory and of buckling problems (Die neuen Hilfstafeln zur Berechnung von Spannungsproblemen der Theorie zweiter Ordnung und von Knickproblemen)

ERNST CHWALLA, *Der Bauingenieur* (Berlin), V. 34, 1959: No. 4, Apr., pp. 128-137; No. 6, June, pp. 240-245; No. 8, Aug. 1959, pp. 299-309 [For errata, see Aug. 1959, p. 309, and Oct. 1959, p. 416]
Reviewed by ARON L. MIRSKY

A thoroughgoing discussion of the underlying theory and application of the "Hilfstafeln für Stabilitätsberechnungen" of the Deutscher Stahlbau-Verbund (Cologne, 1959), this paper is noted here for information purposes, since it pertains primarily to the stability and buckling resistance of steel and similar members loaded as eccentrically loaded struts or columns. Many types of members and structures

are treated, including beam-columns, pony arches, frames and trusses, etc. The "theory of the second order" of the title refers to the analysis which takes into account the influence of the deflections on the moments and stresses (i.e., the second-order terms).

Statics and the engineer

ALFRED PUGSLEY, *The Engineer* (London), V. 210, No. 5459, Sept. 9, 1960, pp. 424-427. *Engineering* (London), V. 190, No. 4924, Sept. 2, 1960, pp. 302-303 (abridged)
Reviewed by ARON L. MIRSKY

The power and beauty of statics, with particular application to a few structural types usually considered as hyperstatic [redundant frameworks (Castigliano's theorem, and complementary energy), suspension bridges, tubes and shells] presented in brief but striking manner. Recommended collateral reading for *all* engineers—mathematicians, too!

Ultimate strength of over-reinforced beams

LADISLAV B. KRIZ and SENG-LIP LEE, *Proceedings*, ASCE, V. 86, EM 3, June 1960, pp. 95-105
AUTHORS' SUMMARY

The flexural strength of rectangular concrete beams controlled by compression is investigated analytically, using stress distribution derived from experimental stress-strain curves. Empirical expression for the ultimate bending moment is presented and the calculated moments of 59 test beams that failed in compression are compared with the ultimate test moments.

Concrete plant standards

Concrete Plant Manufacturers Bureau, National Ready Mixed Concrete Association, *Concrete*, V. 68, No. 6, June 1960, pp. 20-21, 34, 36-37

Describes ready-mixed concrete plant equipment with the object to standardized rated capacities, the basis for determining rated capacities, and certain other features of concrete plant components. The following items of equipment are covered: batchers for aggregates, batchers for cement, batchers for water, bins or silos for aggregate, bins or silos for cement, bins or silos for the separate storage of both aggregate and cement, belt conveyors for aggregate, bucket elevators for aggregate, and bucket elevators and extended in second paper to case of eccentric loading, and tilt(inclination) of plate is investigated. Results are applied to rectangular plates by converting them to equivalent elliptical plates (height-width ratio = ratio of minor-major diameters; and equal areas). Numerical examples are given.

Orthogonal wire net of hyperbolic-paraboloid form under vertical states of loading and temperature changes (Das orthogonale Seilnetz hyperbolisch-parabolischer Form unter vertikalen Lastzustaenden und Temperaturaenderung)

H. K. BANDEL, *Der Bauingenieur* (Berlin), V. 34, No. 10, Oct. 1959, pp. 394-401
Reviewed by ARON L. MIRSKY

The hyperbolic-paraboloid concept is here applied to structures primarily in tension, the result being the stabilized suspended roof. Author develops methods of analysis for determining the forces in and displacements of the wires, under conditions of (1) prestressing of the wires, (2) vertical roof loads, and (3) temperature changes. The effect of horizontal loads is not included.

Matrix formulation of the folded plate equations

A. C. SCORDELIS, *Proceedings*, ASCE, V. 86, ST10, Oct. 1960, pp. 1-22
AUTHOR'S SUMMARY

An analytical procedure, utilizing matrix algebra, is developed for determining longitudinal stresses, transverse moments, and vertical deflections in folded plate structures. The procedure provides an efficient and systematic method for analyzing structures of this type. The sequence of matrix operations can be readily programmed for a digital computer having available matrix subroutines. A sequence which has been programmed for the IBM 704 digital computer is described.

Materials

Use of fly ashes as aggregates or binders in steam cured concrete (Emploi des cendres volantes comme aggregat ou comme liant dans les beton etuves)

R. Ducreux, *Revue des Matériaux* (Paris), No. 533, Feb. 1960, pp. 43-48
Reviewed by Phillip L. Melville

A laboratory study was made of mortars with fly ash as partial replacement for either the fine aggregate, the cement or both. Curing was in saturated air at 80 C. Testing was in compression on cubes. It was found that heating increased strength, that fly ash as an aggregate increased strength and that fly ash as cement replacement considerably increased strength after 4-hr treatment.

Sources of aggregate used in Virginia highway construction

Edwin O. Gooch, Robert S. Wood, and William T. Parrott, *Mineral Resources Report* No. 1, Virginia Department of Conservation and Economic Development, Division of Mineral Resources, 1960, 65 pp.
Highway Research Abstracts
V. 30, No. 9, Oct. 1960

Materials used for highway aggregate in Virginia range from highly metamorphosed rocks of Precambrain age to unconsolidated sand and gravel deposits of recent age. Limestone, dolomite, and quartzite are produced in the Ridge and Valley province. Granite, various types of gneiss, diabase, basalt, marble, limestone, sandstone, and conglomerate are produced in the Blue Ridge and Piedmont provinces. Sand and gravel are produced in the Coastal Plain province.

Aggregate is classified into grades A, B, and C on the basis of three physical properties: abrasion loss, specific gravity, and absorption. Tests used in determining the physical properties are described in the text and the specifications for each grade are listed.

The name and location of each aggregate producer is given in the text and on the accompanying map. The geology of each pit and quarry is described, physical test data on the aggregate are listed with each description, and the size of the operation or rate of production during 1957 or 1958 is given.

Field control of fresh concrete (Controle du beton hydraulique sur les chantiers de betonnage)

A. Lezy, *La Technique Modern-Construction* (Paris), V. 15, No. 1, Jan. 1960, pp. 83-95
Reviewed by Alexander M. Turitzin

An account is given of the research carried out at the Laboratoire Central des Monts et Chaussees with a view of developing a method for testing the quality of fresh concrete in the field.

It is generally conceded that the laboratory study and test of concrete mixes are not sufficient. Some reliable test must be devised for field use. It often happens that under field conditions with available material, equipment, and contractor's organization a different concrete is obtained than the one in the laboratory.

After testing first batches, it would be possible to rectify granulometric mixes, since once concrete is placed it is too late to correct it and the structure should be either accepted or rejected.

Concrete mix water — How impure can it be?

Harold H. Steinour, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 3, Sept. 1960, pp. 32-50
Author's Summary

The effects of chemicals dissolved in mix water on the strength development of mortar and concrete are reviewed. Natural, fresh water rarely contains more than 2000 parts per million (0.2 percent) of dissolved solids, and is generally suitable as mix water. Waters contaminated with industrial wastes, but free of suspended solids, appear also to be generally suitable at these low concentrations. Much larger contents of the impurities generally present in natural waters can also be tolerated, except for the alkali carbonates and bicarbonates, which may have significant effects even at 2000 ppm. Of other inorganic impurities,

of possible industrial origin, ones that may be detrimental at moderate concentrations are the sulfides, iodates, phosphates, arsenates, borates, and compounds of lead, zinc, copper, tin, and manganese. Organic solutes are also suspect, especially sugars. Sea water, although it contains 3.5 percent of dissolved solids, gives concretes of good early strength, but often somewhat reduced later strength. Risk of corrosion of steel limits the use of sea water in reinforced concrete and prohibits its use in prestressed concrete.

Properties of Concrete

Contribution to the problem of freezing of water in building materials (Beitrag zur Frage der Vorgange beim Gefrieren von Wasser in den Baustoffen)

H. Pfeiffer, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 3, Mar. 1960, pp. 114-116

Reviewed by Ferdinand S. Rostasy

The often observed discrepancies between the results of laboratory frost resistance tests and the actual behavior of building material when exposed to weather leads author to a thorough discussion of the freezing and thawing phenomenon. The author states that the German standard test (DIN 52-104 and DIN 105) inadequately imitates the complicated interaction of temperature change, water absorption, and desiccation. Descriptive models are used to discuss the various processes connected with freezing and thawing. Recommendations for an accelerated frost resistance test are given.

Theory of linear viscoelasticity

D. R. Bland, Pergamon Press, Inc., New York, 1960, 125 pp. $7.50

This book formulates the stress-strain relations of and solves various stress analysis problems for linear viscoelastic materials. It starts out with an introduction to the concepts of viscoelasticity making use of one-dimensional models and then proceeds into the foundations of three-dimensional linear viscoelasticity. Treats problems in stress analysis for sinusoidal oscillations, quasi-static problems, and for dynamic conditions. The final chapter deals with model fitting to measured values of complex modulus or compliance since actual materials rarely have a stress-strain law which in differential form contains only a few terms.

Shrinkage and cracking in cements, mortars, and concretes (in Spanish)

M. H. Lafuma, *Revista Societe Cubana Ingenieurs* (Havana), V. 58, No. 4, 1958, pp. 242-266

Ceramic Abstracts
V. 43, No. 5, May 1960

An exhaustive investigation is summarized as follows: Coarsely ground cements shrink less. Ring tests of pure cement pastes showed that shrinkage and cracking depend on the physical, chemical, and mechanical properties of the cement. The two methods used by manufacturers for portland cements (increase in lime content and fineness) give early strength but increase shrinkage and the tendency to crack. In use, portland cements should not for a few days be subjected to stresses above the compressive strength permissible for shrinkage and tendency to crack. Cements for portland cement should be ground to 2500 to 3500 Blaine. Precautions in storing, taking, and preparing samples for tests are discussed, and test curves are added.

On the sedimentation of binding grouts (La sedimentation des coulis des mants)

A. Steopoe and V. Moldovan, *Revue des Matériaux* (Paris), No. 535, Apr. 1960, pp. 83-89

Reviewed by Phillip L. Melville

Effects on cement and concrete of calcium lignosulfonates are discussed. A series of tests was made on cylindrical samples of hardened grout cast vertically. Trend to segregation, variations in specific weight, compressive strength, and chemical composition were studied along the samples Lignosulfonates acted as dispersant and retarder. As the lignosulfonate is increased the homogeneity of the grout increases but specific weight and compressive strength decrease.

Structural Research

Determination of the elastic properties and of the damping of materials and measurement of the rigidity of complex structures (Determination des proprietes elastiques et d'amortissement des materiaux et measure de la rigidite des structures complexes)

PIERRE SORIN and ARMAND GIET, *Le Génie Civil* (Paris), V. 136, No. 19, Oct. 1, 1959, pp. 399-400

Reviewed by ARON L. MIRSKY

Brief note describing modified method of testing specimens and structures by forced vibration, whereby effect of weights of pendulums generating the forced vibration is eliminated.

Dynamic analysis avoids vibration

DIMITRI A. NESTERENKO and RICHARD E. WITTER, *Consulting Engineer*, V. 15, No. 2, Aug. 1960, pp. 96-99

Reviewed by ARON L. MIRSKY

Structural vibrations may become dangerous to structures, to equipment, and even to human beings. Article presents a capsule compendium on the subject: criteria for determining what industrial buildings require vibration analysis (e.g., monolithic reinforced concrete structures with two or more stories), human sensitivity, equipment sensitivity, dynamic design of structures, damping. Recommended, especially as an eye-opener for the noncognoscenti.

Theoretical and experimental investigation of secondary equilibrium configurations of precompressed high beams (in Italian)

E. BRUZZESE, *Ricerca* (Naples), V. 9, Jan.-June 1958, pp. 32-50

APPLIED MECHANICS REVIEWS V. 13, No. 9, Sept. 1960

A straight concrete beam is prestressed by a cable the axis of which is a parabola. The depth of the cross section is large in comparison with the width. There are no external loading forces. In certain cases, if the prestressing force is increased, a critical value of this force is reached at which the plane form of bending becomes unstable, that is, the beam buckles sidewise. This critical value is computed. The instability is due to combined torsion and bending. To control the theoretical result, an experimental investigation was carried out. The model was made of a synthetic resin (Perspex); the prestressing component was a steel chord. Agreement between calculated and measured results was rather good (10 percent).

Experimental investigations with skewed slabs on elastic bearings (Modellversuche mit elastisch gelagerten schiefen Platten)

H. DANIEL and W. RICHTER, *Der Bauingenieur* (Berlin), V. 34, No. 10, Oct. 1959, pp. 406-407

Reviewed by ARON L. MIRSKY

Presents results of an experimental investigation of a slab with length-width ratio of 1.0 and a skew angle of 50 deg, in the form of principal moments at various points for (1) dead load and (2) concentrated (wheel) loads, for both elastically-supported (rubber bearings) and rigidly-supported slabs. For simplicity, the slab was replaced by a grid of 6 x 6 beams, uniformly spaced. It is shown that for certain conditions the behavior of the elastically-supported slab approaches that of the rigidly-supported slabs.

Theoretical and experimental investigations of ferro-concrete beams subjected simultaneously to the action of deflecting and torsional forces (in Russian)

N. N. LESSIG, *Teoriya Rascheta i Konstruir. Zhelezobeton. Konstruktsii* (Theory of the Calculations and Construction of Ferro-Concrete Structures), Gosstroiizdat, Moscow, 1958, pp. 73-84; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6878

APPLIED MECHANICS REVIEWS V. 13, No. 11, Nov. 1960

The experiments carried out made it possible to indicate possible ways in which breakdown could occur in ferro-concrete beams of rectangular section, working under deflection and torsion and to derive calculation formulas for the determination of the carrying capacity of such beams.

General

Building and construction terms

BENJAMIN LIPOWSKY and MURRAY BERSTEN, Arco Publishing Co., Inc., New York, 1960, 57 pp., $3

A picture dictionary and guide to building terminology. In the first section, conventional building and construction terms are given with a brief definition. Second section consists of full diagrams giving the grouping of building terms by "family" so that by using the two sections, the term is defined both by words and by picture. The definitions are based on New York City usage and building laws.

Building research (Bauforschung)

Edited by Deutsches Bauzentrum e. V. Dokumentationsstelle für Bautechnik, Verlag Dr. Rossipaul, Stammheim-Calw., Germany, 1959, 292 pp., 16 DM

A survey of German literature on building research covering 1945-1958. The 2559 titles (published in numerous periodicals and series) are arranged according to the Universal Decimal Classification. An extensive index of authors and subjects makes for easy use of the bibliography.

Some of the subjects covered in the bibliography are precast concrete, reinforced concrete, prestressed concrete, load calculations, stress conditions, safety factors, structural elements, foundations, tunnels, bridges, highways, lightweight concrete, materials, and many others.

Professional engineer's examination questions and answers

WILLIAM S. LA LONDE, JR., McGraw-Hill Book Co., New York, 2nd Edition, 1960, 589 pp., $7.50

Book is primarily designed to give assistance to an applicant for registration as a professional engineer. In addition to containing some helpful general information relative to registration laws and preparation for examination, more than 600 questions are presented from recent actual examinations for professional engineer or land surveyor. Complete answers are given for each question showing the type which examiners will accept and credit. All major fields of engineering are covered in addition to the basic sciences. A list of reference textbooks is also given for study in the various divisions.

List of publications of the U. S. Army Engineers Waterways Experiment Station

U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Jan. 1960, 141 pp.

Lists publications issued to date by the U. S. Army Engineer Waterways Experiment Station and reports of hydraulic model investigations sponsored by other offices of the Corps of Engineers. The publications of the Waterways Experiment Station are grouped under headings corresponding to the primary fields of investigation, namely: hydraulics, soil mechanics, flexible pavements, and concrete. Certain reports on related subjects are also listed. An additional section lists reports that have been published and distributed by the Waterways Experiment Station for other agencies. The publication also tells how publications can be purchased.

Some structural engineering advances

O. A. KERENSKY, *The Engineer* (London), V. 210, No. 5449, July 1, 1960, pp. 4-6
Reviewed by ARON L. MIRSKY

Abstract of some papers at the Sixth Congress of the International Association for Bridge and Structural Engineering, Stockholm, June 27-July 1, 1960. These include "Le barrage de Tourtemagne en Valais (Suisse)," by F. Panchaud, describing a prestressed thin arch dam retaining a 850,000 cu m reservoir subject to emptying and filling with glacial waters at all seasons, and a paper by Prof. Gibschmann describing Russian use of precast reinforced concrete units in composite-action structures, especially as the bottom flange in the compression zone of continuous girders.

ACI NEWS LETTER

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memo from the President:

AROUND THE WORLD

CHARTERED IN 1906 in the United States, the American Concrete Institute has grown in stature and breadth until it now has 10,358 members and 1675 JOURNAL subscribers in 92 other countries. Of these members, 2916 live outside of North America—Canada, Mexico, and the United States. The proportion of members from other countries is believed to be higher than that for any other "American" organization. Truly ACI is international in scope.

Our cosmopolitan membership is not just a statistic, however. It means that the need and value of the magic material to which it is devoted are soundly and universally recognized. Research in Australia, design in South America, and codes in Europe are made known to great numbers working under a wide variety of conditions; and free international exchange is at its possible best. Each has much to learn from the other, and many have much to contribute.

How far would prestressing be used today in the United States if it had not been preceded, actually triggered, by the remarkable advances in Europe? And American methods have undoubtedly fed back useful applications and techniques. Other examples could be cited of ideas crossing national boundaries, back and forth.

Recognizing the value of internationality, the American Concrete Institute is doing everything possible to foster it. The Board of Direction has established a Committee on International Relations, from which many fruitful ideas are

expected. ACI has representatives on a collaboration committee with the Comité Européen du Béton. We have sent delegates abroad to meetings of that organization (CEB), the International Association of Bridge and Structural Engineers, the International Congress of the Precast Concrete Industry, the International Federation of Prestressed Concrete, and others. The Institute is a member of RILEM, the international association of research and testing laboratories.

ACI has been active in various projects concerned with the translation and publication of papers and reports.

A cooperative program with the Comité Européen du Béton, Cement and Concrete Association in England, and Portland Cement Association has resulted in the translation into English of various CEB reports. These have been published in limited number and made available to researchers in the subjects reported.

The Institute has authorized, as well as published, translations in Spanish of various reports and standards, the most notable being the ACI Building Code, the ACI-ASCE Committee 323 tentative recommendations on prestressed concrete, and ACI's standards on mix proportioning and painting. The Building Code has also been published in German.

The *ACI Manual of Concrete Inspection* has been published in Japanese. The latest edition of the *Concrete Primer* has been translated and published in Japan, Uruguay, and Italy; the earlier edition was published in many other languages.

Many translations and condensations of JOURNAL papers have appeared in the technical press in Argentina, Belgium, Brazil, Denmark, France, Germany, Greece, India, Israel, Italy, Japan, Mexico, Netherlands, South Africa, Spain, Sweden, Thailand, Turkey, Uruguay, and Yugoslavia.

But, publications are only a part of ACI's over-all efforts. The Institute several years ago worked with the Portland Cement Association in producing two films: "How to Make Quality Concrete" and "How to Transport, Place, Finish, and Cure Quality Concrete." While U.S. showings of the films have been made by PCA, ACI has loaned copies for overseas showings in Australia, India, New Zealand, and other places. The Institute also produced a film showing construction of the headquarters building and this too has been shown overseas.

An important part of the work of ACI Committee 115, Research, is the annual compilation of research in progress by various agencies throughout the world. The 1960 compilation included reports from 115 organizations—42 of these being outside the U.S. Organizations outside the United States reporting research work included work going on in many countries: Australia, Canada, Belgium, Denmark, England, France, Germany, India, Italy, Japan, Netherlands, New Zealand, Norway, Portugal, Scotland, South Africa, Switzerland, and Wales. You can well imagine how this has facilitated exchange of information between research workers.

ACI has also cooperated in the International Cooperation Administration American professional societies program whereby persons who have participated in ICA technical training programs in the U.S. can maintain contact with American colleagues and keep abreast of current developments through ACI membership.

The Institute is also one of many organizations participating in the trade missions program of the U.S. Department of Commerce. Some of ACI's publications accompany each trade mission; after the mission the library re-

mains in the country for continuing use.

Some of our technical committee members are located overseas, as well as numerous authors of JOURNAL papers. The JOURNAL also welcomes and publishes news of people and events in many countries.

One of the great opportunities of ACI membership is that of rubbing shoulders with experts, in print or at meetings. The fact that so many of our colleagues are from so many fields in so many countries is a great source of gratification, not only because of the technical benefits but also because of social influences on general international understanding.

Joel W. Kelly
President

Kesler reappointed TAC chairman

Clyde E. Kesler, professor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, was reappointed chairman of the Technical Activities Committee for an additional term of 1 year, by recent Board of Direction action. J. J. Shideler, manager, Products and Applications Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill. and W. J. McCoy, director of research, Lehigh Portland Cement Co., Allentown, Pa., were reappointed members for 2-year terms. A. T. Hersey, director of technical services, Alpha Portland Cement Co., Easton, Pa., was appointed to a 2-year term on the Technical Activities Committee. All appointments are effective Feb. 22, 1961.

Appointment of technical committee chairmen

The Board of Direction has approved the appointment and/or reappointment of the following chairmen of ACI technical committees:

Committee 201, Durability of Concrete in Service: Hubert Woods, director of research, Portland Cement Association, Research and Development Laboratories, Skokie, Ill.

Committee 321, Design of Reinforced Concrete Slabs (joint with ASCE): C. P. Siess, professor of civil engineering, University of Illinois, Urbana.

Committee 328, Limit Design (joint with ASCE): Herbert A. Sawyer, Jr., professor, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge.

Committee 333, Design and Construction of Composite Structures (joint with ASCE): Irwin A. Benjamin, Granco Steel Products Co., Granite City, Ill.

Committee 337, Strength Evaluation of Existing Concrete Structures: Merit P. White, professor and head, Civil Engineering Department, University of Massachusetts, Amherst.

Committee 716, High Pressure Steam Curing: George W. Washa, professor and chairman, Department of Mechanics, University of Wisconsin, Madison.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 215, Fatigue of Concrete
Freeman P. Drew
Association of American Railroads
Chicago, Ill.

Committee 315, Detailing Reinforced Concrete Structures
Melvin W. Jackson
Structural Engineer
Lakewood, Colo.

St. Louis—February 20-23

57th Annual ACI Convention

VERSATILITY KEYNOTES the outstanding 4-day meeting awaiting ACI convention-goers at the 57th Annual Convention, February 20-23, at the Chase-Park Plaza Hotel, St. Louis. The technical program is geared to the interests of the designer, contractor, research man, and others in fields related to the concrete industry. Speakers of national and international prominence will present approximately 20 papers at five scheduled technical sessions.

The first 1½ days will be devoted to technical committee meetings with a "Technical Committee Progress" session on Tuesday afternoon, February 21. Brief committee reports will be presented on work underway and in progress within various committees.

The first general session opens Wednesday morning following addresses by Mayor Raymond Tucker of St. Louis and ACI President Joe W. Kelly, professor of civil engineering, University of California, Berkeley.

Special events

The local planning committee under the chairmanship of Howard A. Coleman, vice-president and general sales manager, Missouri Portland Cement Co., St. Louis, has augmented the fact-filled technical sessions with a number of "extras" to make the 57th ACI annual meeting in St. Louis a highlight of the year.

Twenty-one exhibitors have already booked space at the exhibit to be held in conjunction with the meeting.

Architectural student design competition

A design contest, with prizes totaling $400, will be held in conjunction with the convention, with competition open to architectural students at Washington University, St. Louis, and the University of Illinois, Urbana. The design models and drawings will be displayed in the exhibit room at the convention.

Judges for the contest will be one representative each from ACI, the faculty of Washington University, the faculty of University of Illinois, and a practicing architect from Illinois and one from Missouri. First prize is $250; second, $100; and third, $50. The judges reserve the right to withhold awards if they feel no entries deserve recognition.

Ladies' events

Special features for the ladies include a full program of tours, luncheons, and other entertainment especially planned to suit milady's taste. Featured in the tours will be visits to St. Louis' famed waterfront, the Memorial Plaza, old and new sections of the city, the Climatron, Shaw's Gardens, and the Jefferson Memorial.

An Hawaiian breakfast at one of the city's largest department stores, a tour of another department store, and a luncheon at the convention hotel are scheduled. A dinner, Wednesday evening, February 22, will be preceded by a hospitality hour sponsored by the local committee.

TECHNICAL PROGRAM

TUESDAY AFTERNOON, FEBRUARY 21

Technical Committee Progress Session

TAC Committee on Monographs—G. E. Burnett

Committee 115, Research—S. J. Chamberlin

Committee 116, Nomenclature—D. L. Bloem

Committee 212, Admixtures—B. E. Foster

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete—T. R. Jones

Committee 318, Standard Building Code—R. C. Reese

Committee 321, Design of Reinforced Concrete Slabs—Joint ACI-ASCE—C. P. Siess

Committee 323, Prestressed Concrete—Joint ACI-ASCE—M. Schupack

Committee 335, Deflection of Concrete Building Structures — M. V. Pregnoff

Committee 616, Coatings for Concrete—W. H. Kuenning

Committee 621, Aggregates—W. R. Waugh

Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE—J. R. Janney

WEDNESDAY MORNING, FEBRUARY 22

First General Session

President's Address—Joe W. Kelly, University of California, Berkeley

Report of Meeting of International Association of Bridge and Structural Engineers (Stockholm) — Douglas McHenry, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Report of Third International Congress of Precast Concrete Industry (Stockholm)—Arsham Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D.C.

Report of Tours of Cement and Concrete Plants in Russia—A. Allan Bates, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Report of Autoclaved Cellular Concrete Conference (Gothenburg)—John K. Selden, Autoclave Building Products Association, Toledo, Ohio

WEDNESDAY AFTERNOON, FEBRUARY 22

Design and Construction Session

Prestressed and Preloaded Concrete Pillars—John J. Reed, Colorado School of Mines, Golden; and C. David Mann, St. Joseph Lead Co., Bonne Terre, Missouri

Mitchell Park Conservatory—W. J. Hufschmidt, Hufschmidt Engineering Co., Milwaukee

Analysis of Stresses and the Load Carrying Capacity of the Lincoln Boulevard Bridge over Lost River, Idaho—Reynold K. Watkins, William A. Cordon, and Elliot Rich, Utah State University, Logan

Supporting Structure for Retractable Roof of Pittsburgh Public Auditorium—Edward Cohen, Ammann and Whitney, New York; and H. Rey Helvenston, Public Auditorium Authority of Pittsburgh and Allegheny County, Pittsburgh

Materials Session

Water-Cement Ratio Versus Strength—Another Look—Herbert J. Gilkey, Iowa State University, Ames

Chemical Prestressing of Concrete Elements Using Expanding Cements—Alexander Klein and T. Y. Lin, University of California, Berkeley

How Good Is Good Enough?—E. A. Abdun-Nur, consulting engineer, Denver, Colo.

Load Tests on Patterned Concrete Masonry Walls—R. O. Hedstrom, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

THURSDAY MORNING, FEBRUARY 23

Committee 216 Symposium on Fire Resistance

Introduction—Chairman C. C. Carlson, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Fire Resistance of Reinforced Concrete—Irwin A. Benjamin, Granco Steel Products Co., Granite City, Ill.

Fire Resistance of Building Construction Protected by Concrete—Robert R. Sheridan, Eastman Kodak Co., Rochester, N.Y.

Fire Resistance of Prestressed Concrete—G. E. Troxell, University of California, Berkeley

Design Session

A Controlled-Deflection Design Method for Reinforced Concrete Beams and Slabs—Adrian Pauw and Donald G. Alcock, University of Missouri, Columbia

Theory and Analysis of Prismatic Folded Plate Structures—John E. Goldberg and K. C. Mahmoud, Purdue University, Lafayette, Ind.

Rectangular Concrete Stress Distribution in Ultimate Strength Design—Allan H. Mattock, Ladislav B. Kriz, and Eivind Hognestad, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Tie Requirements for Reinforced Concrete—Boris Bresler and P. Gilbert, University of California, Berkeley

THURSDAY AFTERNOON, FEBRUARY 23

Research Session

Under auspices of Committee 115, Research

This session differs from general convention sessions in that the information is not released for publication. A group of short papers will be presented on research techniques and preliminary results on a number of current projects not yet sufficiently advanced to justify final reports.

EXHIBITORS

Autolene Lubricants Co.
Denver, Colo.

Calcium Chloride Institute
Washington, D.C.

Chicago Fly Ash Co.
Chicago, Ill.

Clipper Manufacturing Co.
Kansas City, Mo.

Detroit Edison Co.
Detroit, Mich.

Expanded Shale, Clay & Slate Institute
Washington, D.C.

Granco Steel Products Co.
St. Louis, Mo.

Walter N. Handy Co.
Evanston, Ill.

Laclede Steel Co.
St. Louis, Mo.

Maginniss Power Tool Co.
Mansfield, Ohio

Johns-Manville
New York, N.Y.

Master Builders Co., The
Division of American-Marietta Co.
Cleveland, Ohio

Missouri Prestressed Concrete Association
St. Louis, Mo.

Leschen Wire Rope Division
H. K. Porter Co., Inc.
St. Louis, Mo.

Rex-Spannall, Inc.
Milwaukee, Wis.

Sika Chemical Corp.
Passaic, N.J.

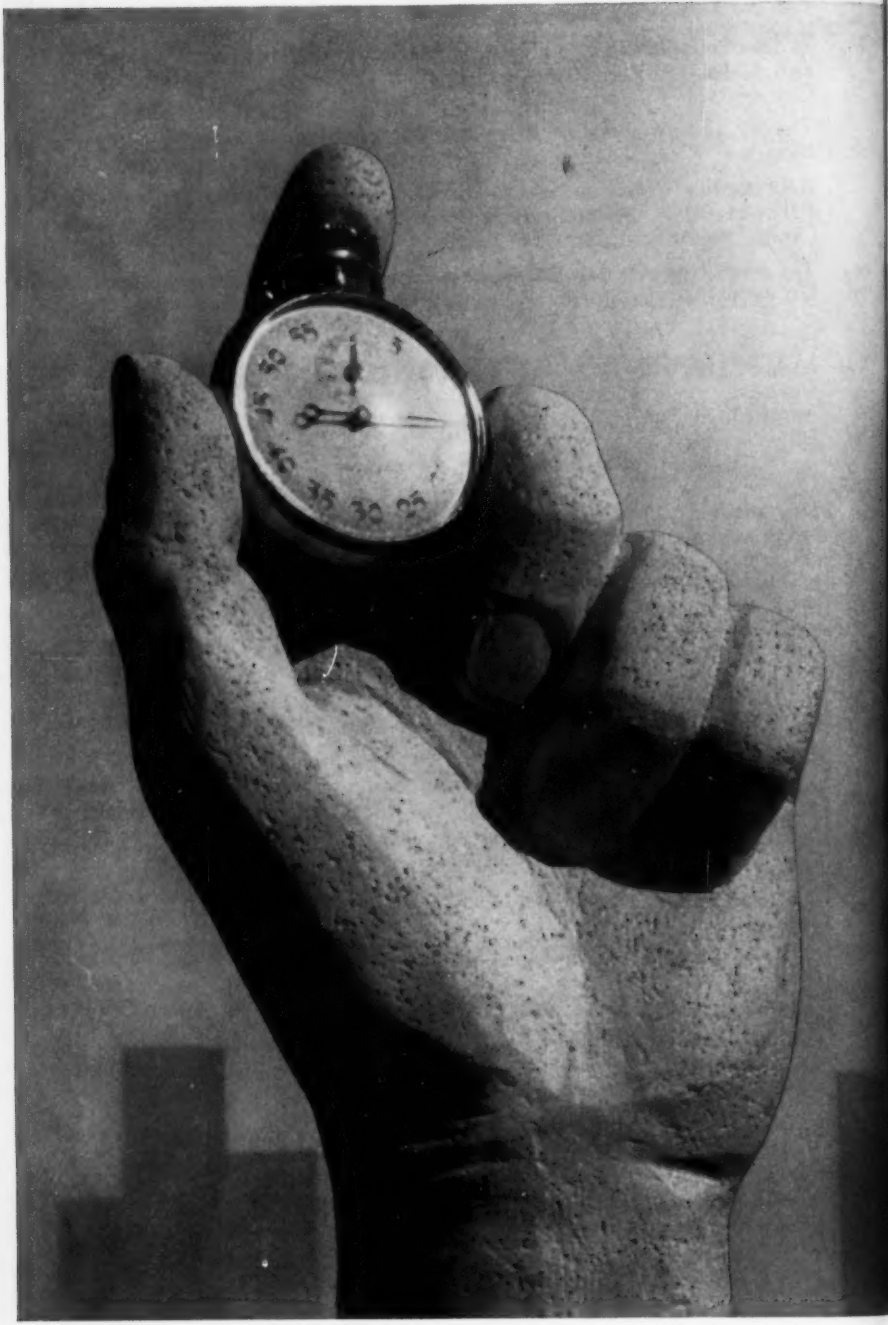
Sonoco Products Co.
Hartsville, S.C.

R. L. Spillman Co.
Columbus, Ohio

West Penn Power Co.
Greensburg, Pa.

Water Seals, Inc.
Chicago, Ill.

Western Waterproofing Co.
St. Louis, Mo.



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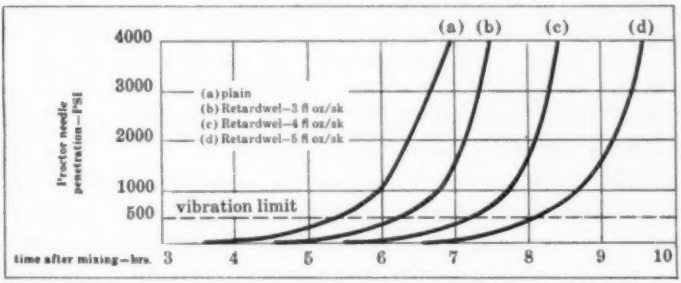
Retardwel®, a product of Johns-Manville, world's largest industrial user of portland cement, is the liquid admixture that controls concrete setting time and prevents premature stiffening due to high temperatures *without* loss of early strength.

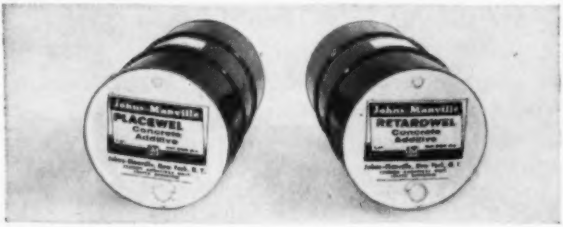
Retardwel permits a reduction in the water required for proper placing and provides all the advantages of concrete fabricated with a minimum paste content. Its use delays the initial set of concrete and provides a slower rate of heat evolution, thereby minimizing thermal stresses. Therefore, the use of Retardwel will aid in eliminating shrinkage cracks. Only 3 ounces of Retardwel per sack of cement will control setting time (see chart) and still increase 24-hour strength as much as 50%.



Retardwel's unique properties will also—

■ Increase durability ■ Improve dimensional stability
■ Increase workability ■ Reduce permeability ■ Increase density

For normal weather conditions, J-M Placewel® is recommended. Johns-Manville's two concrete admixtures have gained the acceptance of architects and engineers throughout the world. For full information write: Johns-Manville, Box 14, New York 16, N. Y.



JM PRODUCT JOHNS-MANVILLE

Celite Division

double-

BREATHTAKING BIRDSEYE VIEW during construction of the exhibition palace shows roof vault scaffolding which was erected on the building's main floor to reduce over-all height needed

BRILLIANT NEW ADORNMENT on the Paris horizon is the Exhibition Palace of the Centre National des Industries et des Techniques, a three-arched ribbed double shell vault that rises higher than the Arc de Triomphe (theoretical rise is 152 ft). Vault openings are closed to the weather with a facade of tempered plate glass supported by a hollow lace work of stainless steel, structurally independent of the roof. Leading from the three wide streets which parallel the sides of the triangular palace are 131 entrances, seven of which can be used by the largest trucks.

Triple the size of the Grand Palais where most Paris exhibitions have been held, the new structure has more than a million square feet of floor area available on five levels for commercial and industrial showings. The Exhibition Palace will also house the nation's first permanent center and library of technical and economic activities and information. This unique building is believed to set two new world records: it has the longest known span for a thin shell vaulted structure, 676 ft at the facade and 780 ft at the groin; and it has the world's largest surface supported at a single point, 75,000 sq ft.

Design problem

Complex structural problems were laid down by the architects and owner when they selected their design requirements from among a number of initial suggestions. The great roof was to be a completely watertight shell structure covering a 226,000-sq ft area in the shape of an equilateral triangle

shell roof without walls

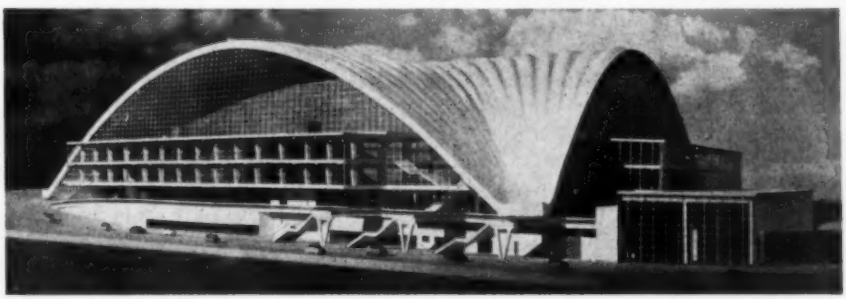
A concrete expression of French engineering creativity . . . in response to architectural demands for a triangular exhibition structure of mammoth proportions. Thin shell roof cast in place in two layers is complemented by floors of precast elements and prestressing in the main floor's composite structure.

with sides 715 ft long. Resting on only three supports at ground level, the triangular shell was to be formed by the intersection of three cylindrical surfaces with horizontal generators. The final design in concrete was chosen from among three reinforced concrete designs, three structural steel designs, and one composite steel-concrete design submitted by several engineering firms and consulting engineers.

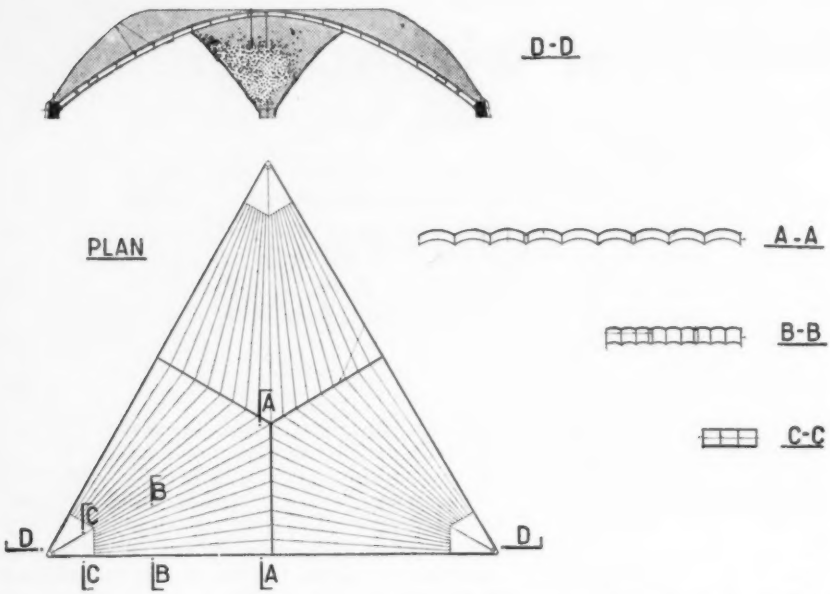
Roof scheme chosen

The reinforced concrete roof is a self-supporting corrugated double shell similar to a groined vault without any structural members such as ribs or structural arches. Flow of forces toward the supports is direct and follows a minimum path to the three abutments at the vertices of the equilateral triangle. On each side of the Y-shaped crown line, the roof consists of nine successive corrugations (see sketch, p. 12) whose projected width decreases from 22½ ft at the crown to 1.85 ft at a distance 43 ft from the springing. Thickness of each of the two shells is only 2.36 in. for the two center sections nearest the crown, remains constant (equal to 2.5 in. for the other sections) down to some 98 ft from the springing. The thickness of the shell slabs then gradually increases to 23.6 in. at the abutment where their width is only 13 ft on either side of the groin.

The two cast-in-place shells are connected by precast reinforced concrete webs. Total depth of the cross section measured along a perpendicular to the centroidal axis is 6.2 ft at the crown, increasing to 9 ft at the springing.



PARIS' NEW ROOF without walls, as it appeared in model stage. Glazed facade and interior floor are structurally independent of the roof



Simplified plan and sections of the unique roof. Principal dimensions are included in text of article

At the crown, where each of the six groups of nine sectors has its maximum width, a specially strengthened transverse member was provided to receive all the forces produced by the different fibers of the shells or webs which abut against it. Ninety-day compressive strength of concrete for this crown wall concrete averaged 6800 psi. Shell concrete had a 90-day ultimate compressive strength of 6100 psi.

Advantages of the double shell

The double shell arrangement concentrates material at the extreme fibers, providing maximum strength in flexure and torsion with a minimum of concrete. There are functional advantages as well; electrical conduits, service walks, and forced air heating pipes are all concealed between the two shell slabs.

The upper shell was cast on cement-impregnated fiberboard forms, rough enough to insure good bond to the concrete, which serve as insulating panels. The double shell construction plus these panels gives a K factor of approximately 0.86 — better thermal insulation than that specified. The same combination also provides good acoustical insulation and prevents condensation on the intrados of the lower shell slab.

Watertightness

The roof is reported to be waterproof because of the compaction of the concrete, which was suitably vibrated, enriched with cement and smoothed at the surface. Since the shell is in compression, cracks are not anticipated—and the second shell offers increased safety against water penetration. Nonetheless, a light colored polyester-based paint was applied to the outside of the upper shell to seal pores, achieve a uniform appearance for concrete of

different ages, and to avoid external damage from industrial atmospheric effects.

All rainwater drains toward the abutments, coming off in torrents as the drainage from 75,000 sq ft of roof area concentrates at a single point. A special discharge arrangement was provided to channel this water into the sewer system.

Abutments, foundations, anchorages

The three massive prismatic abutments, 20 ft high, were designed to insure proper anchorage of tie members and rigid fixing of the shell. The abutments rest on stepped footings foundations extending 20 ft to marl and limestone formations. The abutment concrete has a 90-day compressive strength of 5600 psi.

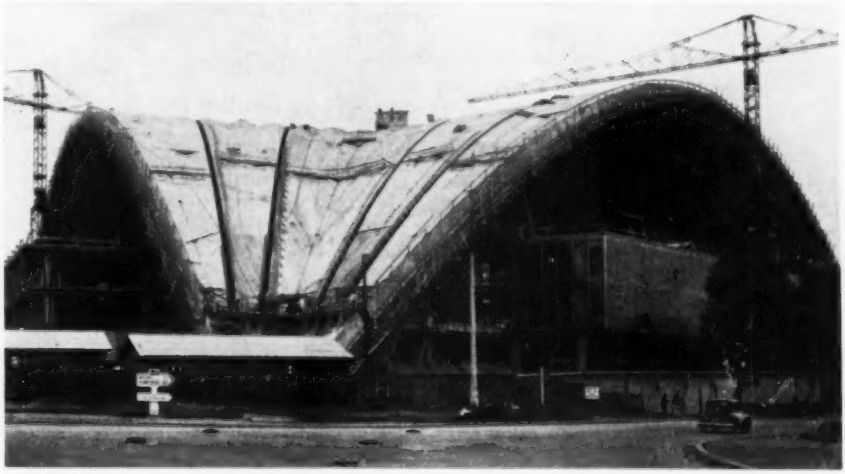
Ties between the abutments in the plane of the facade are formed of underground horizontal and inclined concrete members containing prestressed high tensile steel cables. Underground cable anchorages are concrete pads prestressed in three directions, mounted on shaft piles that extend downward through a layer of limestone.

Precast concrete featured in floors

Much of the floor surface is made of rectangular precast reinforced elements 28 x 4.9 ft, with integral joists cast in place. These slabs rest on main beams spanning 39 to 46 ft, whose lower sections were precast. The beams are supported on truncated pyramid columns.

The 124,000-sq ft main floor area rests on 51 columns of grooved hexagonal cross section 39 ft above the ground level. Columns are placed at the vertices of 59-ft equilateral triangles. This floor is of double slab construction, with the space between slabs (about 7 ft) housing utilities, ducts, conduits, cables, etc. The slabs are composed of precast triangular elements, 20 ft on a side, with main supporting beams cast in place because of their size. The floor is prestressed in three directions at two levels, and the whole functions as a composite structure in reinforced and prestressed concrete.

The upper slab surface of the main floor, with a load capacity of 205 psf, supported scaffolding during construction of the roof, thus reducing the height required for the vault's centering.



Close-up of building during construction

Architects and engineers

Conceived by three principal architects, Robert Camelot, Jean de Mailly, and Bernard-Henri Zehrfuss, the Exhibition Palace was built with private capital at a cost of nearly $12 million. Three civil engineering contracting firms — Balency et Schul, Bousirron, and Coignet — collaborated in executing the unique structure.

Acknowledgments

Information and illustrations in this article came from the Commercial Counselor of the French Embassy, New York; from *France Actuelle*, V. 7, No. 18, Oct. 15, 1958; and from the following three technical papers:

Balency-Bearn, M. A., and Lacombe, M. G., "La Construction du Palais des Expositions du Centre National des Industries et Techniques au Rond-Point de la Défense," Supplement to *Annales de l'Institut Technique du Bâtiment et des Travaux Publics* (Paris), Series: Technique Générale de la Construction (24), No. 127-128, July-Aug. 1958, pp. 877-902 (in French).

Esquillan, Nicolas, "The Design and Construction of the Shell Roof of the Exhibition Palace of the National Center of Industries and Technology, Paris," Cement and Concrete Association, London, Dec. 1958, 26 pp.

Esquillan, N., "Le Palais des Expositions du Centre National des Industries et Techniques. Conception et Exécution de la Voûte Coque," *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. III, No. 6, 1958, Nov.-Dec. 1958, pp. 484-501 (in French).

ACI participates in 11th Southwest Concrete Conference

The 11th Southwest Concrete Conference was held January 11-12 at Oklahoma State University, Stillwater. The conference was arranged by R. P. Witt, professor, School of Civil Engineering, and L. F. Sheerar, Division of Engineering and Industrial Extension, Oklahoma State University.

Kenneth D. Cummins, ACI technical director, participated in the program, speaking on "Formwork for Concrete Structures." His talk was based primarily on the work of ACI Committee 622 on formwork for concrete.

The conference covered a variety of subjects including concrete in civil defense; concrete possibilities in the agricultural field; paving design considerations; color; concrete finishing; epoxy resins; and precast, prestressed concrete construction. Unique methods being used in the construction of the Cowboy Hall of Fame in Oklahoma served as the basis for one of the papers.

Other ACI members participating in the program included Les W. Baxter, general field engineer, northeast Oklahoma district, Portland Cement Association; Raymond J. Schutz, vice-president for research and development, Sika Chemical Corp., Passaic, N.J.; and Rollo D. King, structural field engineer for the Portland Cement Association, Tulsa.

The conference was cosponsored by the American Concrete Institute Oklahoma Chapter, Oklahoma Ready-Mix Concrete Association; National Ready Mixed Concrete Association; National Sand and Gravel Association, and Portland Cement Association.

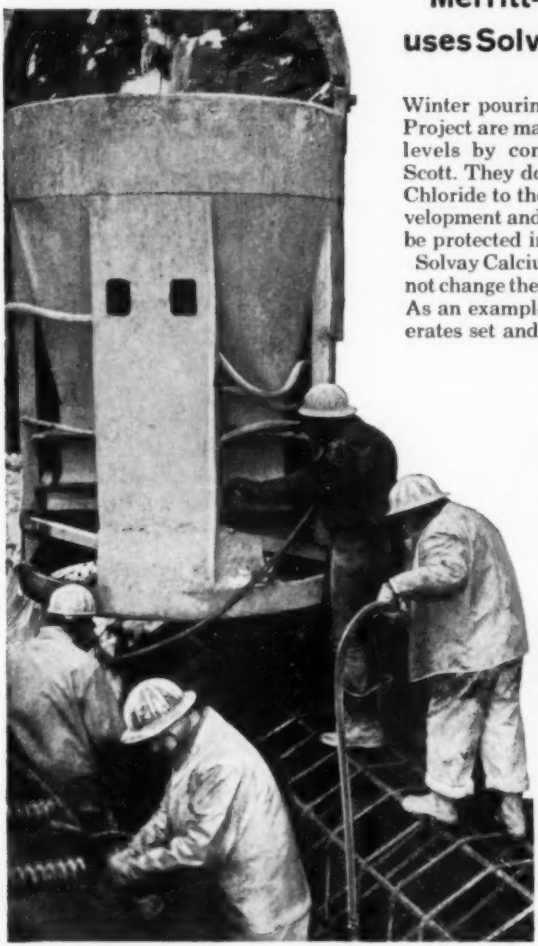
Western building industries exposition planned for 1961

The first all-inclusive exhibit covering construction industries in the West has been announced as a 4-day trade show to take place in Los Angeles, October 7-10.

Sponsored jointly by a group of building material and construction trade associations, long-range plans for Western Building Industries Exposition encompass an annual exhibit serving the 13 western state region, including Alaska and Hawaii, with location rotating from year to year among the four or five major cities of the west.

TO SPEED WINTER CONCRETING AT NIAGARA POWER PROJECT

Merritt-Chapman & Scott uses Solvay Calcium Chloride



Winter pouring schedules at Niagara Power Project are maintained at near warm weather levels by contractor Merritt-Chapman & Scott. They do it by adding Solvay® Calcium Chloride to the mix to accelerate strength development and shorten the time concrete must be protected in freezing weather.

Solvay Calcium Chloride accelerates but does not change the basic action of portland cement. As an example, at 32°F a 2% addition accelerates set and development of early strength . . . 50% less protection time is needed. Ultimate strength is 2% to 12% higher, too.

Get full facts about the benefits of Solvay Calcium Chloride for concrete work. Write for Solvay's free, 38-page technical booklet, "The Effects of Calcium Chloride on Portland Cement."

Here Solvay Calcium Chloride in the amount of 1% by weight of the cement is added to heated water used to mix winter concrete at Niagara Power Project. Total concrete used on this project to construct intake, power plants and conduits will be more than 3.5 million cubic yards . . . enough to build a 6-lane expressway from Chicago to Cleveland. When completed Niagara will be capable of generating more than 2,000,000 kilowatts of power, making it the largest hydro-electric development in the Western World. It is being built by the New York State Power Authority.

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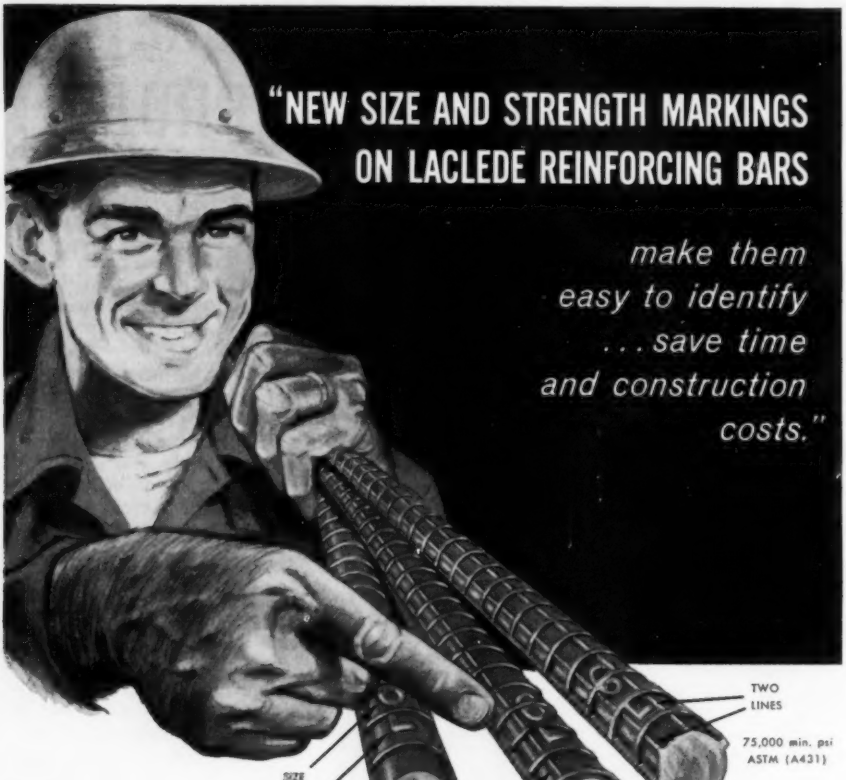
"NEW SIZE AND STRENGTH MARKINGS ON LACLEDE REINFORCING BARS
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SIZE
MANUFACTURER
33,000, 40,000, 50,000 psi ASTM (A15)
ONE LINE
60,000 min. psi ASTM (A432)

Pick up any Laclede Multirib reinforcing bar. A rolled-in marking immediately tells you its size, strength and manufacturer. Laclede Multirib bars of high strength steel are rolled to conform to the two new ASTM standards: A-432 for 60,000 psi minimum yield point steel, and A-431 for 75,000 psi minimum yield. These bars can be used with assurance under the Ultimate Strength design methods included in the new A.C.I. building code. Bars of either 60,000 or 75,000 psi yield strength are identified by one and two longitudinal ribs, respectively.

These high-strength steels can contribute a saving of as much as 15% in the total construction cost of a concrete structure. The worker in the field can quickly find the right bar specified for the job; with the grade of steel well identified, many inspection and laboratory test charges may be eliminated.

For your next construction job, specify these time-saving, money-saving Laclede Multirib reinforcing bars.

Visit our Booth — #48 at the A. C. I. Meeting in St. Louis, Feb. 20-23.

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SAINT LOUIS, MISSOURI ◆ Producers of Steel for Industry and Construction

Positions and Projects

Northern California Chapter to contribute ACI 55-Year Index to educational institutions

The ACI Northern California Chapter has a project underway to present a copy of the *ACI 55-Year Index* to each of the 38 northern California universities, state colleges, junior colleges, and private colleges offering complete or near-complete engineering courses.

Each copy will have a bookplate on the inside cover noting that the index was presented with the compliments of the Northern California Chapter, American Concrete Institute. It is tentatively planned to present other ACI publications in the future.

Michigan ACI Chapter awards

In conjunction with its monthly luncheon meetings, the ACI Michigan Chapter has originated an award for the speakers—the presentation of an ACI lapel pin. The award is presented to the main speaker at the end of each meeting. (It might be added that ACI headquarters is studying the possibility of making lapel pins available to the membership.)

The lapel pin has also been awarded on special occasion—for example, a unique contest was a feature of the November meeting.

T. E. M. Wheat, chief structural engineer on Eberle M. Smith Associates, Inc., Detroit, was guest speaker. He spoke on the design techniques and construction of both the structural model and the actual roof of the cafeteria at the Henry Ford Community College, Dearborn, Mich., currently under construction by Bryant and Detwiler, Detroit general contractors.

The shell-like structure is a doubly curved surface, 4 in. thick, spanning an area 57 ft square, such that typical sections cut through the roof perpendicular to each other are identical sine curves, one being its own depth above the other.

Because of the appearance of the roof, sort of a "bumpy" slab, a contest was held among those present at the meeting to determine a name for the structure. The winner was F. F. Widrig, chief engineer for American Prestressed Concrete, Inc., who suggested calling the roof a "sinaloid." Mr. Widrig was awarded an ACI pin for his semantic victory after which, uniquely, he joined the ACI.

Southern California Chapter meets with ASTM

The ACI Southern California Chapter meeting held January 19, featured a joint meeting with the Southern California District Council of American Society for Testing Materials.

A. Allan Bates, vice-president of research and development, Portland Cement Association, Chicago, who holds a high place in the official ranks of both ASTM and ACI, was featured speaker for the meeting.

Dr. Bates is ASTM president and is currently a member of the ACI Board of Direction and of the Board's new Committee on Chapter Activities. Dr. Bates spoke on the subject of "The Role of Material in Shaping the American Economy."

Oklahoma Chapter announces Muskogee short concrete course

ACI Oklahoma Chapter and the Portland Cement Association are cooperating with Oklahoma State University, Stillwater, in presenting a Practical Course in Quality Concrete, beginning February 14, in Muskogee, Okla.

Les W. Baxter, general field engineer for the northeast Oklahoma district of PCA, is program coordinator for the

short course. All of the lectures will be given by experienced engineers from the ACI Oklahoma Chapter.

The course will cover fundamentals of quality concrete; design of concrete mixtures; air entrainment, admixtures and field testing. Other subjects to be discussed in the course include: transporting, placing, finishing, curing of concrete, maintenance, and special concrete problems.

The course is planned for construction, ready-mixed concrete, testing and inspection personnel; engineers, architects, contractors, home builders, and highway personnel, and will be conducted in five weekly, 2-hr sessions.

Muskogee is the second of the series of short concrete courses to be given throughout Oklahoma during the coming year; the first was held in October, 1960, in Bartlesville.

Northern California Chapter holds joint meeting with ASTM

The first meeting of the ACI Northern California Chapter of 1961 was a joint sessison with the Northern California District, American Society for Testing Materials. The meeting was held at the Engineer's Club, San Francisco, on January 23, with an overflow attendance. A. Allan Bates, vice-president of research and development, Portland Cement Association, Chicago, currently president of ASTM, was featured speaker for the meeting. Dr. Bates is a member of the ACI Board of Direction.

The chapter wound up its 1960 schedule of meetings at a dinner meeting at the Golden Bull Restaurant in Oakland on December 1. Paul Klieger, manager, Field Research Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., presented an illustrated lecture on "Production and Control of Concrete," during which he led the group through the fundamentals of testing and control up the ladder through quality control, statistical analysis, and significance of test results. His audience was highly receptive and there was a rather lengthy discussion period at the conclusion of his talk.

During 1960 the chapter meetings have been held in San Francisco, Sacramento, and Oakland, the Bay area being the geographical center of the section.

Precast concrete tolerances topic of Michigan Chapter meeting

Tolerances for precast concrete structural shapes evoked lively discussion at the Detroit December meeting of the ACI Michigan Chapter.

Robert E. Beerbower, division manager, Michigan Flexicore Division of Price Brothers Co., Inc., Livonia, was the featured speaker. He presented suggested standard tolerances for precast concrete structural shapes. These suggestions were drafted by three precast concrete manufacturers in Michigan: Price Brothers Co.; American Prestressed Concrete, Inc., Centerline; and Precast Industries, Inc., Kalamazoo. The suggested tolerances were presented to chapter members for critical discussion with the thought that comments would aid the industry in developing a set of tolerances applicable to Michigan construction.

The suggested tolerances, Mr. Beerbower noted, were intended for columns, beams, double tees, single tees, hollow core slabs, solid slabs, and miscellaneous shapes in both standard and special forms. It was noted that not only over-all dimensions, but also location of openings and special cast-in items would be governed by the tolerances.

The tolerances submitted for discussion by Mr. Beerbower were:

	Thickness, in.	*Width, In.*
Under 7 in.	± 1/8	± 1/8
7 in. to 16 in.	± 3/16	± 3/16
16 in. to 36 in.	± 1/4	± 1/4
Over 36 in.	—	± 3/8

Continued on p. 22

Chicago Tribune Photo

World's Largest Water Filtration Plant Uses 513,000 Yards of Fly Ash Concrete

By *far* the largest project of its kind, Chicago's new Central District Filtration Plant is using over half a million cubic yards of concrete!

And every yard is made from a mix employing Fly Ash to help produce the finest concrete for the purpose.

Here is mass concrete in long walls, huge "rooms," tunnels, ducts, floors, covering slabs and superstructures. Much is exposed to extremes of weather. Much is constantly submerged.

This concrete must literally "hold water." Beyond the required structural strength, its resistance to water penetration is perhaps the most important quality called for. Fly Ash in the mix is proved to aid greatly in making a concrete that passes rigorous hydrostatic tests.

Much of the placing here is done by the Pumpcrete method, for which Fly Ash, with its very fine fines and ball-bearing-shaped particles, is a tremendous help. In this application, as in many others, 28-day strengths of Fly Ash concrete were equal to, or greater than, comparable concrete without Fly Ash. Proper mix design is the answer.

We invite you to visit Booth 47 at the ACI Convention. Each of the companies below has technical data and competent engineers to help you in designing the most effective mixes employing cement and Fly Ash.

WALTER N. HANDY COMPANY, INC.
P. O. Box 549, Evanston, Ill.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

WEST PENN POWER COMPANY
Cabin Hill, Greensburg, Pa.

McNEIL BROTHERS, INC.
P. O. Box 4015, Bridgeport 7, Conn.

CHICAGO FLY ASH COMPANY
228 N. La Salle St., Chicago 1, Ill.

CONCRETE PERFORMANCE REPORT:

Monolithically-cast POZZOLITH concrete lift slabs speed construction of new "twin" apartment buildings

LIFT SLAB

The new twin 15-story Huron Towers apartments are the tallest to be built in the U.S. with the Youtz-Slick system. The slabs—lifted *two-at-a-time* with 36 hydraulic jacks—are the largest, measuring 215 ft. x 69 ft. x 9 in. and weigh 830 tons each. On the Huron Towers project, each floor was cast *monolithically . . . without joints or "pour strips" of any kind.*

Specifications originally called for a minimum of 6.5 bags of cement for 4000 psi concrete. Preliminary tests, however, showed that the *specified strength could be maintained most economically* by using 5 bags of cement and POZZOLITH . . . and provided the desired reduction in shrinkage that allowed construction of these large slabs without pour strips. This mix design was submitted, approved and used throughout the Huron Towers project. All tests exceeded specified strength.

The contractor developed a unique sequence of casting the 15,000 sq. ft. slabs at floor level, working alternately from one building to the other on a 3-day cycle. While one crew placed reinforcement and conduit in one building, the crew in the other building was placing concrete . . . pumped through an 8-inch line from a centrally-located pump.

POZZOLITH concrete was used throughout both buildings for (1) *ease of pumping* . . . there were no plugged lines during the entire pumping operation (2) *virtually no honeycombing* . . . despite intricate reinforcing steel and electrical and mechanical work in the slabs (3) *minimized shrinkage and cracking* . . . inspection after lifting showed no cracks through the slab.

LOWER BASEMENT FLOORS in both buildings were cast on the ground to serve as casting beds for the liftable slabs. The ground floor and upper basement floor slabs are 14" plus 2½" joist-type slabs spanning onto wide flat beams of the same depth. The 12 apartment floors and the roofs are flat slabs, all 9" thick except the first living floor, which is 10".

HURON TOWERS, Ann Arbor, Michigan, was designed by King and Lewis Architects, Inc., with structural engineering by R. H. McClurg Associates, Inc.—both of Detroit. Project Manager was B. R. Sloan and Project Engineer, Willard C. Reeder—both of Long Construction Company, General Contractor, Kansas City, Mo. Killins Gravel Co. of Ann Arbor supplied the POZZOLITH ready-mix concrete.

TOWERS

MAXIMUM LIFTING HEIGHT of roof slab for this building was 145 ft. For its "twin"—142 ft. The number of liftable slabs—15, in this building, sets a record as the greatest number of concrete decks to be lifted in the United States. Total floor area of the two apartment buildings is 465,000 sq. ft.—431,000 of which were lifted.

Master Builders field men worked closely with the engineer, contractor and ready-mixed concrete producer on the Huron Towers job to aid in achieving the common goal of uniform, high-quality concrete at lowest cost-in-place.

No matter how large, how small, how unique or how conventional the project, call in the local Master Builders field man. He will show you how concrete produced with POZZOLITH is a more durable, versatile building material . . . superior in performance, quality and economy to plain concrete or concrete produced with any other product.

Write for your free copy of the detailed "Huron Towers Concrete Performance Report".

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World-wide manufacturing and service facilities

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MASTER BUILDERS. POZZOLITH*

*POZZOLITH *is a registered trademark of The Master Builders Co. for its ingredient for concrete which provides maximum water reduction, controls rate of hardening and increases durability.*

	Length, in.
Under 20 ft	± 1/4
20 ft to 30 ft	± 3/8
30 ft to 40 ft	± 1/2
40 ft to 50 ft	± 5/8

Over 50 ft ....1/8 in. $\times \frac{\textit{length in ft}}{10}$

Camber or sweep

Permissible variation in camber or sweep to be:

$$3/8 \text{ in.} + 1/8 \text{ in.} \times \frac{\textit{length in ft}}{10}$$

Variation in camber between adjacent and abutting units to be one-half the total variation.

Ends out-of-square

Where the dimension is 12 in. or less: 1/32 in. per in. of width or depth

Where the dimension is over 12 in.: 3/16 in. + 1/64 in. per in. of width or depth

The chapter has invited written discussion on the suggested tolerances and plans to devote the May meeting to this subject.

Highway officials from 50 states meet in Detroit

Nearly 2000 highway officials from throughout the United States and Mexico attended the 46th annual meeting of the American Association of State Highway Officials in Detroit, November 28-December 2.

The 5-day meeting included numerous executive and technical committee meetings, field trips, and approximately 15 technical sessions.

Two papers presented at the design session which were of special interest to ACI members were: 'The Development of Pavement Performance — Design Relationship of AASHO Road Test," William N. Carey, Jr., chief engineer for research, AASHO Road Test, Ottawa, Ill., and "Discussion of AASHO Recommended Guides for the Design of Rigid and Flexible Pavement Structures," T. S. Huff, Texas State Highway Depart, Austin, Chairman of the AASHO subcommittee on pavement design practices.

The materials session featured a program of motion pictures covering materials used in road building. The films covered airfield pavement construction; reinforced bituminous concrete overlays; and joint sealers and concrete adhesives for modern highways.

Topics covered at the construction session included new construction equipment in use; control of construction to secure specified surface smoothness of bridge decks; methods for field testing of densities and moisture content of earthwork, base and surface courses; and advances in methods for compacting earthwork, base and surface courses.

Graduate CE scholarships available in 1961

A survey of graduate scholarships, fellowships, and assistantships available to civil engineering students in 1961 has been prepared by Chi Epsilon, national civil engineering honor fraternity. Copies of this detailed report of opportunities for advanced study have been sent to the civil engineering departments of all American and some foreign colleges and universities. The graduate programs include over 500 individual awards whose value totals some $1 million. Single copies of this survey are available upon request from Michael A. Spronck, Chi Episilon Fraternity, Martinsville, N.J.

AED annual meeting

The 42nd annual meeting of the Associated Equipment Distributors will be held February 5-9 in Los Angeles, Calif. Some 3000 machinery company executives from throughout the United States and Canada are expected to converge for the 5-day meeting.

AED has lined up a distinguished array of speakers from all areas of the construction business to analyze 1960's problems and to chart a course for the industry to follow in 1961.

ACI Library Aided by Gilkey

The ACI technical library was materially strengthened by the recent addition of 41 volumes of *Transactions*, American Society of Civil Engineers, plus the following titles, all donated by Herbert J. Gilkey, professor of theoretical and applied mechanics, Iowa State University, Ames.

Cements, Mortars and Concretes, Myron S. Falk, 1904, 176 pp.

Reinforced Concrete, A. W. Buel and and C. S. Hill, Engineering News Publishing Co., New York, 1904, 434 pp.

Concrete-Steel, W. Noble Twelvetrees, Whittaker and Co., London, 1905, 218 pp.

Concrete and Reinforced Concrete Construction, Homer A. Reid, Myron C. Clark Publishing Co., New York, 1907, 884 pp.

"Tests on Plain and Reinforced Concrete, Series of 1906," Morton O. Withey, *Bulletin* 175, University of Wisconsin, 1907, 66 pp.

Manual of Reinforced Concrete and Concrete Block Construction, Charles F. Marsh and William Dunn, Archibald Constable & Co., Ltd., London, 1908, 290 pp.

Reinforced Concrete — A Manual of Practice, Ernest McCullough, Cement Era Publishing Co., Chicago, 1908, 128 pp.

"Tests on Plain and Reinforced Concrete, Series of 1907," Morton O. Withey, *Bulletin* 197, University of Wisconsin, 1908, 136 pp.

Der Eisenbetonbau, E. Mörsch, Verlag von Konrad Wittwer, 1908, 376 pp.

Cement Laboratory Manual, L. A. Waterbury, John Wiley & Sons, New York, 1908, 122 pp.

"Test on Bond Between Concrete and Steel in Reinforced Concrete Beams," Morton O. Withey, *Bulletin* 321, University of Wisconsin, 1909, 64 pp.

Principles of Reinforced Concrete Construction, F. E. Turneaure and E. R. Mauer, 2nd Edition, John Wiley & Sons, New York, 1909, 430 pp. (Also 3rd Edition, 1919)

A Treatise on Masonry Construction, Ira O. Baker, 19th Edition, John Wiley &. Sons, New York, 1909, 746 pp.

Concrete - Steel Construction (Der Eisenbetonbau), Emil Mörsch, Engineering News Publishing Co., New York, 3rd Edition (Translation), 1910, 368 pp.

From the Raw to Finished Product, Chicago Portland Cement Co., 1911, 68 pp.

"Test of Bond Between Concrete and Steel," Duff A. Abrams, *Bulletin* 71, Engineering Experiment Station, University of Illinois, 1913, 240 pp.

Materials of Construction, G. B. Upton, John Wiley & Sons, Inc., New York, 1916, 328 pp.

Building Code, City of New York, 1916, 278 pp.

Concrete Plain and Reinforced, Frederick W. Taylor and Sanford E. Thompson, John Wiley & Sons, Inc., New York, 3rd Edition, 1916, 886 pp.

Reinforced Concrete Design Tables, M. Edgar Thomas and Charles E. Nich-

Now . . . the long-awaited

Vol. 2 of GUYON'S

PRESTRESSED CONCRETE

Covers statically indeterminate prestressed concrete structures, dealing with elastic design with special reference to theory of the concordant cable. Second half is on ultimate-load design and theory of the plastic hinge, including a valuable section on structural behavior of prestressed slabs. Gives experimental data and worked-out examples. 1960. 741 pages. $16.75. See also VOL. 1, with revision supplement. 1960. 559 pages. $16.25

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440 Park Avenue South, New York 16, N.Y.

ols, McGraw-Hill Book Co., Inc., New York, 1917, 210 pp.

Estimating Concrete Buildings, Clayton W. Mayers, Aberthaw Construction Co., Boston, 1920, 52 pp.

Concrete, Its Manufacture and Use, Koehring Co., Milwaukee, 1921, 208 pp.

"Standard and Tentative Methods of Sampling and Testing Highway Materials," Bulletin 949, U. S. Department of Agriculture, 1921, 98 pp.

'Comparative Tests of New Billet Steel and Rerolled Steel Reinforcing Bars," Jesse B. Kommers, *Bulletin*, University of Wisconsin, Engineering Series 1252, V. 9, No. 3, 1924, 32 pp.

"Tentative Specifications for Portland Cement Concrete Pavements," *Proceedings*, American Society for Municipal Improvements, 1924, pp. 373-398.

Proposed Specifications for Portland Cement Concrete Pavements, American Society for Municipal Improvements, 28 pp.

"Cement and Concrete," *Bulletin* 18, Kansas City Testing Laboratory and Industrial Testing Laboratory, 1925, 18 pp.

"Portland Cement Concrete of Unusual Character," *Bulletin* 23, Kansas City Testing Laboratory, 18 pp.

Volumetric Changes in Portland Cement Mortars and Concretes Due to Changes Other than Variations in Temperature, Raymond E. Davis, 1927, 22 pp. Reprint from Congres International Pour L'Essai des Materiaux, Amsterdam, Sept. 12-17, 1927.

Atlas Handbook on Concrete Construction, Atlas Portland Cement Co., New York, 1927, 154 pp.

"Tentative Standard Methods of Sampling and Testing Highway Materials," *Bulletin* 1216, U. S. Department of Agriculture, 1928, 132 pp.

Standards of Design for Concrete, No. 3yb, U.S. Navy Department Bureau of Yards and Docks, 1929, 216 pp.

"Tests of Plain and Reinforced Haydite Concrete," F. E. Richart and V. P. Jensen, reprint from *Proceedings*, ASTM, V. 30, Part II, 1930, 34 pp.

"The Effect of Frequency of Vibration in Making Concrete Beams," M. O. Withey, reprint from *Proceedings*, Highway Research Board, V. 15, 1935, pp. 217-233.

Plain Concrete, Edward E. Bauer, McGraw-Hill Book Co., New York, 2nd Edition, 1936, 364 pp.

Special Cements for Mass Concrete, U.S. Bureau of Reclamation, 1936, 230 pp.

"Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity Ratio of Concrete," Vernon P. Jensen, *Bulletin* 44, University of Illinois, 1943, 62 pp.

The historical significance of many of the older works is further enhanced by the signature of A. N. Talbot, a pioneer in the field of concrete testing and design. The Institute is deeply indebted to Professor Gilkey for these volumes. It is hoped that other ACI members will follow his example and expand the ACI library by donation of historical or current works dealing with concrete.

EXHIBIT

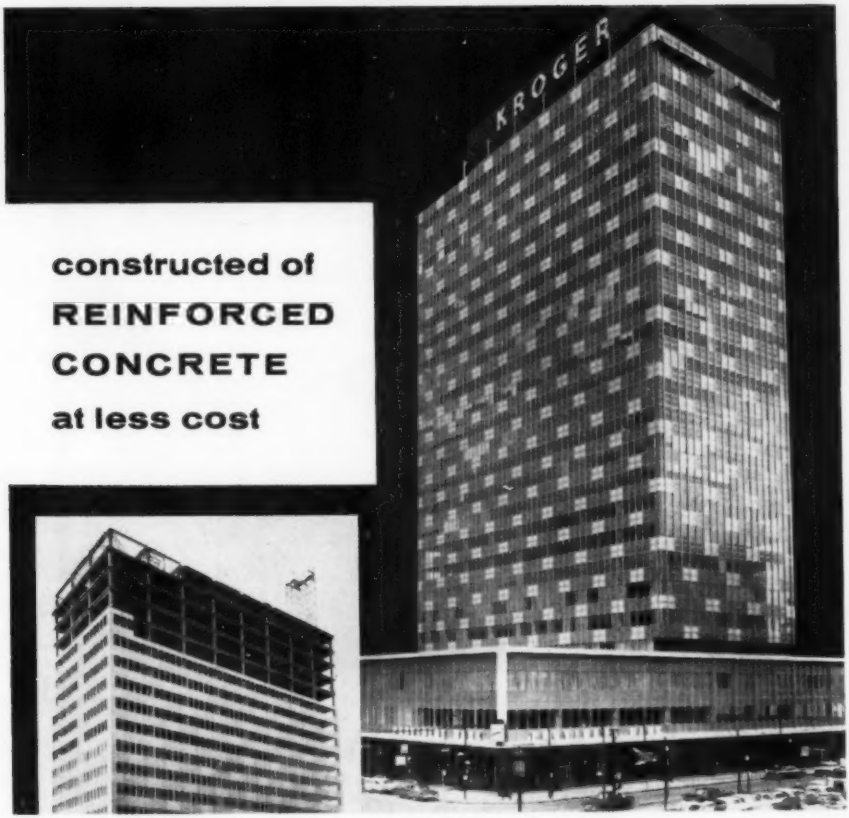
SPACE STILL AVAILABLE FOR 57TH ANNUAL ACI CONVENTION

•

FEB. 20-23, 1961
Chase-Park Plaza Hotel
St. Louis, Mo.

•

Write for application forms today to:
Mr. Paul Magoon
ACI Exhibits Chairman
915 Olive, Room 913
St. Louis 1, Missouri



constructed of REINFORCED CONCRETE at less cost

Kroger Building, Cincinnati, Ohio
Owner: Milner Enterprises, Inc., Jackson, Mississippi
Architects and Engineers: Hedrick & Stanley, Ft. Worth, Texas
Contractor: George A. Fuller Company, New York and Dallas

Another outstanding example of reinforced concrete construction is the new $8,000,000, 27-story Kroger building in Cincinnati. Here, architects changed the design to reinforced concrete framing and concrete joist floor construction to reduce over-all construction costs and construction time.

For every type of building, reinforced concrete offers many costsaving and timesaving advantages. To reduce costs, it utilizes most efficiently both reinforcing steel and concrete to provide the most rugged structural frame possible. Construction time is reduced through the availability of all materials from local sources—construction can begin tomorrow and structural changes made without costly delays.

Before you build any type of building investigate the more economical, timesaving, and design advantages of reinforced concrete construction.

Concrete Reinforcing Steel Institute
38 South Dearborn Street, Chicago 3, Ill.

9-6

Addition to Diamond's Department Store, Park Central Shopping Center, Phoenix, Arizona. Associated General Contractors: Bell Vista Properties and E. F. Hargett & Co. Planners: John Graham, Architects and Engineers. Supervising Architect: John Schotanus

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In any project calling for round concrete columns, save time, labor and money with Sonoco SONOTUBE Fibre Forms. There's a type to meet any job requirement . . . in sizes 2" to 48" I. D. Specify length or order standard 18' lengths.

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RILEM International Symposium to be held in Prague in 1961

The 15th session of the Permanent Committee of RILEM (Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions) will take place in Prague on July 30 to Aug. 6, 1961. An International Symposium on the Durability of Concrete will be held and will concentrate on the following points: (1) theoretical questions of concrete durability; (2) mechanics of concrete-texture deterioration; (3) durability tests and control; (4) composition of durable concretes; and (5) experiences with concrete structures.

The symposium is being prepared within the framework of RILEM by the Czechoslovak Academy of Sciences, Institute of Theoretical and Applied Mechanics, Prague.

Master Builders forms firm in Japan

The Master Builders Co., Cleveland, has established a new firm that will manufacture and market Master Builders Products in Japan and other far eastern countries.

The new firm known as Nisso Master Builders Kabushiki Kaisha is a joint venture between The Master Builders Co. and the Nippon Soda Co., Ltd., Japan.

Headquartered in Tokyo, the firm will manufacture and market the full line of Master Builders products throughout Japan, Burma, Thailand, China, Taiwan, South Korea, Okinawa, Laos, Cambodia, and Viet Nam.

ACI member Charles Lyon, formerly special assistant to the vice-president for marketing, has been appointed vice-president of NMBKK. A veteran of nearly 20 years with Master Builders, he served as manager of the Detroit branch office prior to his appointment last year in the marketing division, working out of the Cleveland office.

Levy elected NSA president

Edward C. Levy, president, Edward C. Levy Slag Co., Detroit, Mich., was elected president of the National Slag Association at its 43rd annual meeting in Washington, D.C.

George W. Lanier, vice-president and general manager, Houston Slag Materials Co., Houston, Tex., was elected vice-president; W. S. Shaw, vice-president Munsey Trust Co., Washington, D.C., treasurer; and E. W. Bauman, Washington, D.C., managing director-secretary.

Brabant named V-P of British Columbia Cement

B. M. Brabant has been appointed vice-president and general manager of British Columbia Cement Co., Ltd., Vancouver, B.C. He was also appointed vice-president and a director of the parent organization, Ocean Cement and Supplies Ltd., as well as director of Evans, Coleman and Evans Ltd., and Evans, Coleman and Gilley Bros. Ltd.

In his new appointment he succeeds B. Franklin Cox, who has returned to England to assume other duties with Associated Portland Cement Manufacturers Ltd.

Dundee Cement names two to vice-presidencies

The election of two key executives of Dundee Cement Co., Dundee, Mich., to vice-presidencies by the board of directors was recently announced by Roblee B. Martin, president.

Raymond S. Chase, sales manager, has been appointed vice-president in charge of operations.

Mr. Chase, a former executive of the Masonite Corp. was named sales manager of Dundee in June, 1959. Mr. Ostberg was plant manager of the Alexandria Portland Cement Co. in Egypt when he accepted appointment as plant manager at Dundee in May, 1959.

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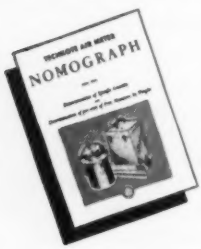

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Ward awarded Athlone fellowship

Michael Allan Ward, a field representative in the Winnipeg branch office of The Master Builders Co., Ltd., has been awarded the Athlone fellowship from the United Kingdom government. He leaves immediately for 2 years study and training in Great Britain.

Mr. Ward intends to devote the first year of his study program at the University of London studying prestressed concrete design and the second year with a control designer of prestressed concrete products.

Mr. Ward received his civil engineering degree in 1959 from the University of Manitoba, Winnipeg, writing his thesis on the subject of concrete technology. He was the recipient of the University Scholarship Award and the Athlete of the Year Award.

In addition to his ACI membership, Mr. Ward is a member of the American Society for Testing Materials, the Engineering Institute of Canada, and the Association of Professional Engineers of Manitoba.

Mr. Ward joined Master Builders in 1958. He will take a leave of absence returning to the firm in the summer of 1961.

Landis elected president of Engineers Joint Council

James N. Landis, vice-president, Bechtel Corp., San Francisco, was elected president of Engineers Joint Council, a national federation of 21 engineering societies, at a recent meeting of the Council's board of directors.

Mr. Landis, who is past present and present council member of the American Society of Mechanical Engineers, succeeds Augustus B. Kinzel, vice-president of Union Carbide, who headed EJC for the past year.

George E. Holbrook, vice-president of E. I. du Pont de Nemours & Co., was reelected vice-president of the council.

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San Diego State College seeks bound ACI volumes

Sanford H. Stone, professor-in-charge of civil engineering, San Diego State College, San Diego, Calif., has inquired as to whether someone might be in a position to donate old bound volumes of the ACI *Proceedings* to their college library. He particularly would like *Proceedings*, V. 51, 52, and 53 or older.

The college will pay the shipping charges on any bound volumes which may be acquired. Contact Professor Stone before forwarding bound volumes.

California holds 13th highway conference

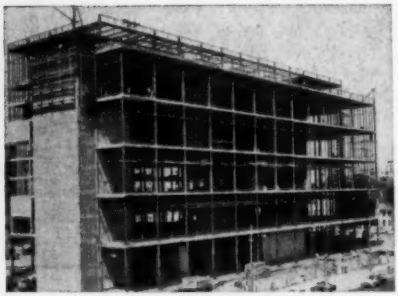
The University of California's Institute of Transportation and Traffic Engineering presented the 13th annual California Street and Highway Conference, January 26-28, on the university's Berkeley campus.

Specialists participating in the program covered various phases of policy, management, and research. Technical developments were covered in 15 group discussions.

Whitcraft retires

Lewis N. Whitcraft, director of research, The Wait Associates, Inc., New York, for more than 12 years, retired December 31.

One of the pioneers in concrete construction, Mr. Whitcraft served the Portland Cement Association in an engineering capacity from prior to World War I until he joined Wait Associates in 1948. Immediately prior to joining Wait, he had been district engineer for



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J-21

PCA in New York City following a term of service in the same capacity in Philadelphia.

Bletzacker named to ASTM committee

Richard W. Bletzacker, supervisor of building research, Ohio State University's Engineering Experiment Station, Columbus, has assumed office as secretary of Committee E-6 on Methods of Testing Building Construction of the American Society for Testing Materials. During his 4 years as supervisor of building research, he has directed extensive fire resistance and structure strength research.

Michigan State schedules concrete conference

A 1-day conference on "Structural Concrete" will be held at the Kellogg Center, Michigan State University, East Lansing, Mich., February 15. The conference will be sponsored jointly by the Civil Engineering Department of Michigan State University and Precast Industries, Inc. of Kalamazoo, Mich.

The objective of the conference will be to explore the practical aspects of new and current trends in structural concrete through the media of four talks covering: prestressed concrete structures and structural members; prestressed concrete manufacturer's problems; ultimate and limit design concepts; and shapes in structural concrete.

The featured speaker of the day will be Edward K. Rice of T. Y. Lin and Associates, Van Nuys, Calif.

TI's Willson elected ESCSI president

Cedric Willson, vice-president, Texas Industries, Inc., Ft. Worth, Tex., has been elected president of the Expanded Shale, Clay and Slate Institute.

Mr. Willson is currently a member of the ACI Board of Direction. He is

a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete; Committee 331, Structures of Concrete Masonry Units; and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Other officers elected at the annual meeting were: Leroy A. Thorssen, Consolidated Concrete Industries, Ltd., Calgary, Alta., Canada, first vice-president; Ben F. Park, Buildex, Inc., Ottawa, Kan., second vice-president; James M. Chandler, Chandler Materials Co., Tulsa, Okla., treasurer; and Lester R. Kennedy, Light Aggregates, Inc., Rapid City, S.D., secretary.

Headquarters for the ESCSI are maintained in Washington, D.C., with Frank G. Erskine the managing director.

Cement plant instrumentation discussed at ISA conference

The Instrument Society of America conducted a Winter Instrument-Automation Conference and Exhibit in St. Louis, January 17-19.

Three technical papers were presented at the cement and lime instrumentation session on January 17. J. C. Andrews, Ideal Cement Co., Denver, spoke on instrumentation and automation of rotary cement kilns, giving a brief history of rotary kiln instrumentation with emphasis on recent developments. John P. Puckett, Leeds and Northrup Co., Philadelphia, Pa., spoke on instrumentation and automatic control techniques as applied to rotary kilns, covering current activities in cement and lime industries with reference to control techniques being applied. H. W. Hilker, Bailey Meter Co., Monterey Park, Calif., covered the discussion of meters and controls for automated operation of rotary kilns, applicable to both dry and wet process installations.

The cement and lime instrumentation session was one of 12 similar technical sessions conducted during the meeting.

LOOKING AHEAD

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

Feb. 5-9, 1961—42nd Annual Meeting, Associated Equipment Distributors, Los Angeles, Calif.

Feb. 14, 1961—Practical Course in Quality Concrete, Oklahoma State University in cooperation with ACI Oklahoma Chapter and the Portland Cement Association, Muskogee, Okla.

Feb. 15, 1961—Structural Concrete Conference, Michigan State University, Kellogg Center, East Lansing, Mich.

Feb. 20-22, 1961—Annual Convention, National Concrete Products Association, Seigniory Club, Montebello, Que., Canada

Feb. 20-23, 1961—57th Annual Convention, American Concrete Institute, Chase-Park Plaza Hotel, St. Louis, Missouri.

Feb. 25-Mar. 3—Engineering Conference, The Institution of Engineers, University of Melbourne, Melbourne, Australia

Mar. 13-16, 1961 — 53rd Annual Convention, American Concrete Pipe Association, Hotel Americana, Bal Harbour, Fla.

Mar. 24-25, 1961—Annual Convention, Midwest Ready Mixed Concrete Association, Drake Hotel, Chicago

May 15-16, 1961 — 41st Annual Meeting, Society of American Military Engineers, Washington, D.C.

May 16-18, 1961—1961 Spring Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

Fearon joins Richmond staff

Frank G. Fearon has been appointed manufacturer's representative for the Ohio, Indiana, and Michigan sales area of the Richmond Screw Anchor Co., Inc., New York.

Sawyer elected PCA board chairman

H. A. Sawyer, chairman of the board and president, Lone Star Cement Co., New York, has been elected chairman of the board of directors of the Portland Cement Association.

Mr. Sawyer, who succeeds Cris Dobbins, president of Ideal Cement Co., Denver, as chairman, has been actively engaged in PCA affairs for several years, serving as a member of the board of directors since 1955, and as a member and chairman of several important PCA committees since 1956.

Five new directors were also named: L. E. Bayer, president of National Cement Co., Birmingham, Ala.; R. M. Craigmyle, chairman of the board of Giant Portland Cement Co., Philadelphia; H. R. Schemm, president of Huron Portland Cement Co., Detroit; Charles E. Shearer, president and chairman of the board of Keystone Portland Cement Co., Philadelphia; and Paul Sunderland, chairman of the board of Ash Grove Lime and Portland Cement Co., Kansas City, Mo.

Five directors were re-elected to an additional 2 year term on the association's board. They are Richard A. Grant, president, Arizona Portland Cement Co. and California Portland Cement Co., both of Los Angeles; Robert E. Pflaumer, president of American-Marietta Co., Chicago; R. D. Raff, president, Diamond Portland Cement Co., Middle Branch, Ohio; Ellery Sedgwick, Jr., president, Medusa Portland Cement Co., Cleveland, and W. S. Ziegler, president of Saskatchewan Cement Co., Ltd., Regina, Sask., Canada, and executive vice-president of Inland Cement Co., Ltd., Edmonton, Alta., Canada.

ACI *Proceedings* on microfilm

Institute members and JOURNAL subscribers may obtain complete ACI JOURNAL, V. 30, July 1958 through June 1959 (*Proceedings* V. 55), including discussion and indexes, on microfilm for $6.15.

Proceedings V. 46 (September 1949 to June 1950) through *Proceedings* V. 54 (July 1957 through June 1958) are also available.

For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

Flicker elected to NSC membership committee

Leonard R. Flicker, director of safety, Permanente Cement Co., Oakland, Calif., has been elected to serve on the membership committee of the cement and quarry section of the National Safety Council.

Calaveras lets contract for construction project

A $390,850 contract for construction of an adit and raise at the quarry of its new cement manufacturing plant now under construction near Redding, Calif., was awarded to Frazier-Davis Construction Co., St. Louis, Mo., by Calaveras Cement Co., a division of The Flintkote Co.

The raise is a 490-ft vertical shaft, 8 ft in diameter, which will serve as an ore pass from the quarry in the Grey Rock hills northeast of the Calaveras plant. The adit is a 700-ft tunnel, 10 ft in diameter, which will connect with the raise. When the quarry is in operation, limestone will be dropped down the vertical shaft and transported to the plant on 6700 ft of belt conveyors starting in the tunnel.

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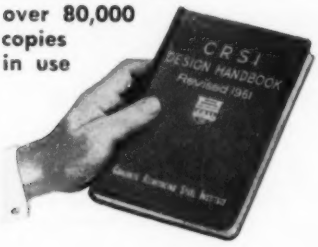

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ASTM Committee Week held in Cincinnati

Thirty of the 85 main technical committees of the American Society for Testing Materials scheduled meetings during ASTM Committee Week in Cincinnati, January 30-February 3.

Highlight of the meeting was a symposium on fire test methods sponsored by ASTM Committee E-5 on Fire Tests of Materials and Construction. A. F. Robertson, National Bureau of Standards, Washington, D.C., presided. Papers presented at the symposium included: "Early History of Fire Endurance Testing in the United States," Harry Shoub, National Bureau of Standards; "A Treatise on Theoretical Fire Endurance Rating," T. Z. Marmathy, National Research Council of Canada; "The New Beam Furnace at PCA and Some Experience Gained from Its Use," C. C. Carlson and P. J. Tatman, Portland Cement Association; and "Surface Flammability of Materials—A Survey of Test Methods and Comparison of Results," J. A. Willson, Factory Mutual Engineering Division.

Committee C-12, Mortars for Unit Masonry; Committee C-15, Manufactured Masonry Units; and Committee D-4, Road and Paving Materials, were among the technical committees which met during the week.

Maginniss establishes new sales department

Maginniss Power Tool Co., Mansfield, Ohio, has formed a new sales department to develop closer relations with its distributors.

Roger A. Minnich, vice-president has been appointed sales manager. Dale E. Anderson has assumed the newly created position of assistant sales manager.

Earl J. Felt

Earl J. Felt, manager of the Transportation Development Section of the Portland Cement Association Research and Development Laboratories, Skokie, Ill., died on November 13 at the age of 52.

Mr. Felt's engineering career included 4 years with the Minnesota Highway Department engaged in pavement and foundation studies; 3 years with the U. S. Forest Service; and 20 years with PCA. He was well known for his laboratory studies and his many published papers dealing with highway engineering, particularly in the field of soil-cement.

Mr. Felt was an active member of ACI, American Society of Civil Engineers, Highway Research Board, and U.S. National Council on Soil Mechanics and Foundation Engineering. He was also active in committee work of the American Society for Testing Materials, and was a member of the honorary scientific society, Sigma Xi. He was a graduate of the University of Minnesota.

PCA holds thin shell seminar in Los Angeles district

The Los Angeles district of the Portland Cement Association presented a seminar of five weekly lectures on the analysis and design of thin shell concrete roofs during October and November.

The lectures included theory and design of the hyperbolic paraboloid, folded plates, and barrel shells. The principle purpose of this seminar was to show that there is a direct technique in analyzing thin shell structures. Design examples were presented to show the methods of analysis.

Robert E. Tobin and James E. Amrhein, structural specialists of PCA, presented the seminar. More than 800 architects and engineers from the area attended the lectures.

Honor Roll

Jan. 1 - Dec. 31, 1960

Today ACI membership has reached an all-time high. Thousands of persons working in the concrete industry throughout the world need ACI services and ACI needs the best of their thinking. Tell your colleagues about ACI activities! Add your name to the Honor Roll next month by signing up a new member.

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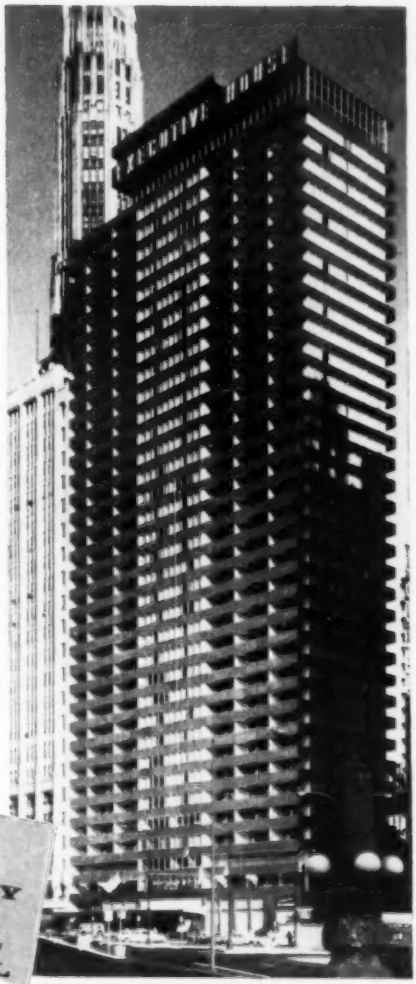
—From article by Mr. Henry Miller, ACI member and president of Miller Engineering Co., Chicago, on p. 215, Sept. issue of "ACI Journal"



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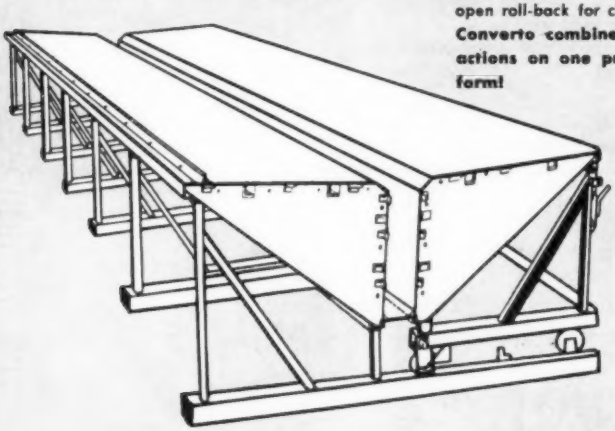

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P. C. Richardson 3
M. A. Rodriguez V. 3
S. Rodriguez 3
W. Rosch 3
H. C. Rose 3
H. J. Rosenberg 3
C. Ross 3
H. Rusch 3
L. W. Ruth 3
S. Sami 3
R. S. Sandhu 3
E. A. Sanford 3
H. A. Sawyer, Jr. 3
J. B. Scalzi 3
M. Schupack 3
H. M. Schwartz 3
W. C. Schwenger 3
A. C. Scordelis 3
J. W. Scott 3
E. Seif 3
H. J. Sexton 3
M. Sherman 3
M. F. A. Siddiqui 3
A. L. Small 3
A. Sofan S. 3
H. M. Stoll 3
F. Stolle 3
W. E. Sullivan 3
F. K. Taskin 3
S. Thaulow 3
H. E. Thomas 3
I. R. Thomas 3
W. A. Thomas 3
J. A. Thomen 3
J. P. Thompson 3
J. C. Thoms 3
A. Tobias 3
E. M. Twining 3
J. T. Tyndall 3
A. Q. Uranga 3
D. A. Van Horn 3
E. Veral 3
E. S. Vieser 3
S. Walden 3
E. H. Walker 3
L. A. Wallis 3
C. Weber 3
A. Weiner 3
L. T. Willoughby 3
C. Willson 3
J. A. Wineland 3
E. C. Wong 3
T. W. Wood 3
M. R. Woodward 3
M. Adawy N. 2½
E. M. Yehya 2½
J. S. Alagia 2
A. H. Andersen 2
C. Asturias P. 2
C. O. Baird, Jr. 2
P. F. Barnard 2
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R. J. Conlon 2
C. M. Crosier 2
A. J. Duncan 2
G. Earle 2
J. R. Fernandez C. 2
A. A. Filos, Jr. 2
C. D. Funnye 2
C. Garcia-Reyes M. 2
E. Garcia-Reyes 2
M. O. Glew 2
E. J. Going, Jr. 2
J. B. Gribbin 2
R. B. Harris 2
G. Hernandez 2
W. W. Hotaling, Jr. 2
E. L. Howard 2
R. A. Iliya 2
P. J. Infante J. 2
C. B. Johnson 2
R. F. Jones 2
K. K. Kirwan 2
T. Lamb 2
B. Lamberti 2
O. Latorre M. 2
L. M. Legatski 2
G. H. Matchette, Jr. 2
H. R. May 2
J. Michalos 2
H. P. Mittet 2
A. Moreno P. 2
A. V. Moreno 2
H. Muguruma 2
R. R. Neal 2
C. L. Nebreda 2
C. I. Oritz 2
A. Ostapenko 2
R. E. Pallais 2
R. M. Patel 2
R. E. Peacock 2
A. Phimister 2
H. Ramadan 2
L. G. Restrepo S. 2
D. Reyes-Guerra 2
S. J. Richelew 2
J. H. Rizo 2
A. W. Ross 2
W. D. Rudeen 2
G. J. Schaumburg 2
R. Schofield 2
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M. Sittenfeld 2
E. L. Spencer 2
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T. Bauer 1
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B. Bresler 1
K. D. Callahan 1
C. Diaz-Gomez 1
R. A. DePasquale 1
E. W. Durling, Jr. 1
R. D. Gipps 1
R. Hernandez G. 1
S. C. Hollister 1
W. J. Hufschmidt 1
A. R. Johnson 1
P. H. Kaar 1
F. Kerekes 1
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M. Mateos 1
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F. Munilla 1
J. Papahiu K. 1
D. Perez F. 1
L. E. Perez O. 1
R. H. Perez S. 1
C. J. Posey 1
M. P. Poucher 1
T. O. Reyhner 1
E. A. Roa 1
J. J. Rodriguez 1
W. W. Sanders, Jr. 1
F. O. Slate 1
M. A. Sozen 1
S. Thompson 1
A. M. Tovar 1
R. G. Tremari 1
M. D. Vanderbilt 1
M. P. White 1

New Members

The Board of Direction approved applications in the following categories: 63 Individual, 13 Junior, and 17 Student, making a total of 93 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,358.

INDIVIDUAL

AHMAD, NAZIR, Rawalpindi, West Pakistan (Engr., Armed Forces Corps of Engrs.)

BENSON, ROGER D., Minneapolis, Minn. (Struct. Engr., Fegles Constr. Co., Inc.)

BLOMDAHL, THORILD W., Montreal, Que., Canada (Chf. Engr., A. Johnson & Co. (Canada) Ltd.)

BOUGHEY, NORMAN R., Guayaquil, Ecuador (Conc. Foreman, Cia. Raymond, S. A.)

BRADLEY, D. E., Kansas City, Mo. (Sales Mgr., Master Bldrs. Co.)

BRILL, BERNARD M., Springfield, Mass. (Rsch. Chem., Shawinigan Resins Corp.)

BROOKS, R. W., Lansing, Mich. (Engr., Banta-Brooks, Inc.)

CHERSON, JAIME, Vedado, Havana, Cuba (Struct. Engr.)

DANZER, RONALD E., St. Louis, Mo. (C. E. III)

DIAS, RUDOLPH STEPHEN, Pittsburgh, Pa. (Des. Engr., Swindell-Dressler Corp.)

EDGERTON, ROY C., Salem, Ore. (Rsch. Engr., Ore. State Highway Dept.)

ESAKOF, EDGAR ARNOLD, Johannesburg, S. Africa (Struct. Engr.)

ESTRADA, JULIO, Caracas, Venezuela (Des. Engr., Min. de Obras Publicas)

FLEISCHMANN, ALBERT C., St. John's, Que., Canada (Cons. Engr.)

GAAFAR, IBRAHIM, Cairo, Egypt, U. A. R. (Asst. Prof., Cairo Univ.)

GHULAM ALI, RAZIA (Mrs.) Karachi, Pakistan (Owner, Pakistan Cemt. Pipe & Constr. Co.)

GORDON, CLARENCE T., Lynn, Mass. (Struct. Engr., Stone & Webster Engrg. Corp.)

GORDON, PAUL, Chicago, Ill. (Owner, Paul Gordon, Struct. Engr.)

GRONQUIST, CARL H., New York, N. Y. (Part., Steinman, Boynton, Gronquist & London)

HART, WILLIAM P., W. Seneca, N. Y. (Constr. Supv., Edward J. Fuhrmann & Co., Inc.)

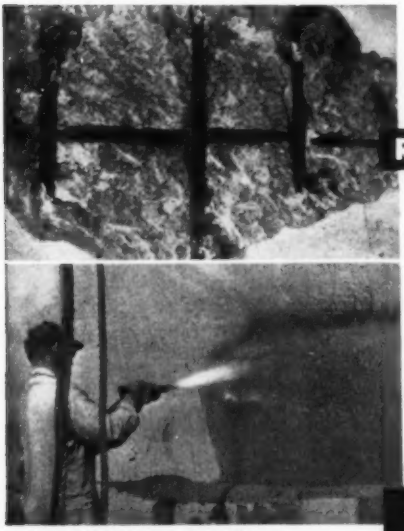
HAYASHI, ALAN S., Honolulu, Hawaii (Des. Engr., Prescon of the Pacific, Ltd.)

HOLOMA, SERGIO, Caracas, Venezuela (Des. Engr., Min. of Pub. Wks.)

HUANG, KE-HSIUNG, Taipei, Taiwan (Struct. Engr., Taiwan Power Co.)

HUSSEINI, AHMED A., E. Ghor Canal Auth., Jordan (Asst. Constr. Engr.)

JERYN, GEORGE, Toronto, Ont., Canada (Struct. Engr., Alex Tobias & Assocs.)



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SURATI, J. N., Toledo, Ohio (Struct. Engr., R. C. Reese & Assocs.)
WILSON, KEITH E., Ames, Iowa (Engr.in-Trng., Iowa State Hwy. Comm.)
WILSON, WALLACE ESTILL, Atlanta, Ga. (Teaching Asst., Ga. Tech.)

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EL-AMIN, A., Andover, Mass. (M. I. T.)
HARLAN, MAURICE R., Austin, Tex. (Univ. of Tex.)
LOPEZ C., ALFREDO, Cali, Colombia (Univ. del Cauca)
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MASS, MARK A., Skokie, Ill. (Ill. Inst. of Tech.)
PAGANINI, OMAR A., New York, N. Y. (Columbia Univ.)
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SANDOVAL LL., HARRY, Cali, Colombia (Univ. del Cauca)
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TSO, STEPHEN H., Berkeley, Cal. (Univ. of Cal.)
VINER, JOHN G., Champaign, Ill. (Univ. of Ill.)
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IMPORTANT NEW PUBLICATION

"An Introduction to High Strength Steel Concrete Reinforcing Bars"

This comprehensive new booklet, published by American Iron and Steel Institute's Committee of Concrete Reinforcing Bar Producers, contains useful information and illustrated examples on the application of high strength steel deformed reinforcing bars, savings in materials and dead load effected by their use, a detailed discussion of welding, copies of available specifications. Tables and diagrams throughout.

For a free copy write to:

Committee of Concrete Reinforcing Bar Producers
American Iron and Steel Institute
150 East 42nd Street
New York 17, New York

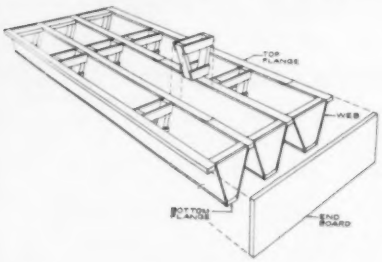
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Plywood building component

Plywood Fabricators Service, Inc. has United States manufacturing rights for Trofdek, a structural building component capable of 50-ft clear spans.

Trofdek is an application of the folded plate idea in which thin sheets of plywood are glued to light lumber stiffeners in a series of miniature troughs, producing a roof



or floor component with extreme light weight. It has been used for roof structures, floors, and concrete forms in more than 500 buildings in Great Britain and Canada.

The system's basic construction is the same for all spans although the depth of the troughs will vary (see drawing). The troughs are fabricated on a 16-in. module. A special spline joint is used to join the 4 ft wide panels on the job. The component is said to be capable of carrying up to 100 times its own weight. Two men can easily handle a standard 100 sq ft panel, although the material combines both framing and deck.

Trofdek may be used as either removable concrete forming or as a permanent part of

the construction. In either case, reinforcing steel may be laid directly in the troughs.

Photo shows concrete forms used on the Canadian Asbestos warehouse and office in Ville St. Laurent, Quebec, Canada, lined with reinforcing steel before placing concrete. Trofdek was used to cover more than 28,000 sq ft in two applications on this job.

The appearance of Trofdek from below is an important factor in its wide acceptance its developers claim. Where the component is used alone or left in place after serving as formwork, a wood ceiling with an unusual, pleasing texture is said to result. Where formwork is removed the concrete takes on the Trofdek shape. Technical data, design information, and construction details are available. —Plywood Fabricators Service, Inc., 3500 E. 118th St., Chicago 17, Ill.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital LETTERS. They may be requested directly from the manufacturers listed below.

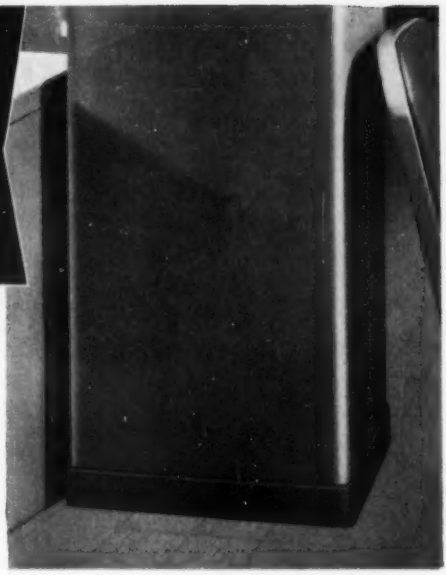
ADHESIVES PROPERTIES CHART — Adhesives properties chart containing latest information on Furane epoxy adhesives, their application, curing times and physical properties. Takes the guesswork out of material selection — Furane Plastics Inc., 4516 Brazil St., Los Angeles, Calif.

UNIVERSITY OF HOUSTON COLLEGE BUILDING — Six-page folder describes how the use of precast concrete cellular floor and roof decks provides a network of raceways for electrical, television, and telephone wiring in the University of Houston's new Fred J. Heyne Building. — The Flexicore Co., Inc., 1932 E. Monument Ave., Dayton, Ohio

ADVANCED PROCESSING EQUIPMENT (Bulletin G560)—Sixteen-page bulletin illustrates and describes dryers, calciners, kilns, coolers, coaters, spheroidizers, flash drying equipment, and other specialized machinery. Includes more than 75 photographs, as well as numerous drawings and diagrams.—Edward Renneburg & Sons Co., 2639 Boston St., Baltimore, Md.

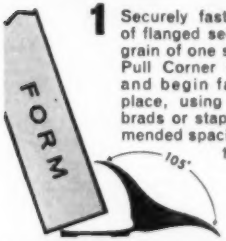
PULVO-MATIC ROCK REDUCER — Bulletin describes and pictures the Pulvo-Matic rock crusher with only one moving part. Its cast manganese-steel rotor cage works in conjunction with manganese-steel liner plates to reduce rock products to graded sizes. Engineering details are given for three crusher sizes.—The Frog, Switch and Manufacturing Co., P.O. Box 431, Carlisle, Pa.

FOR Rounded Concrete Corners



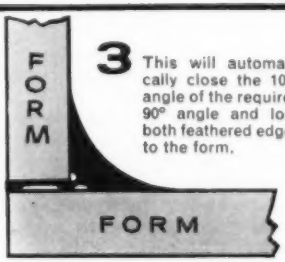
SERVICISED re-usable "GREEN STREAK" Corner Former

SIMPLE INSTALLATION



1 Securely fasten top end of flanged section to end grain of one side of form. Pull Corner Former taut and begin fastening in place, using strip nails, brads or staples. Recommended spacing not more than 4" apart.

2 After Corner Former is fastened, place next side of form in position and tie by conventional methods to form the corner joint.



3 This will automatically close the 105° angle of the required 90° angle and lock both feathered edges to the form.

Servicised "Green Streak" Corner Former provides a quick, easy and low cost method of forming perfect 1" radius rounded corners on piers, beams and all outside corners of poured concrete. Made of a tough, durable and resilient plastic, the Corner Former is simply fastened to the form (see installation drawings at left) and it automatically assumes the correct radius. Because it is readily removed and re-installed, Servicised "Green Streak" Corner Former is by far the lowest cost method of round corner forming you can use.

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NON-METALLIC MINING EQUIPMENT (Bulletin G-11)—Bulletin pictures and describes ten different types of mining and processing equipment made by Fuller to increase automation in the nonmetallic mining and manufacturing industries. — Fuller Co., Catasauqua, Pa.

METAL LATH AND PLASTER SOLID PARTITIONS WITH CHANNEL STUDS (Bulletin TB-5-1)—Collection of pertinent comparative data on solid metal lath and plaster partitions are compiled in two-page technical bulletin.—Metal Lath Manufacturers Association, Engineers Building, Cleveland 14, Ohio

CEMENT ADDITIVES (Bulletin CA-6A) — Two J-M liquid admixtures for concrete, Placewel for water-reduction, and Retardwell for control of setting and water reduction, are described in a folder. Charts and tables give data from tests.—Johns-Manville, Cement Additives Unit, 22 East 40th St., New York 16, N.Y.

IDEAL TRANSPORT STORY — 16-mm color film shows Ideal Cement Co.'s 5½-mile overland conveyor system at Ada, Okla., from land surveys through construction to full operation. 22 minutes. Available for free showing. — Link-Belt Co., Dept. LBN, Prudential Plaza, Chicago 1, Ill.

MAGIC-VULC CAULKING COMPOUNDS — Technical bulletin describes a line of Magic-Vulc Thiokol caulking compounds, #450 Series, for the building construction industry.—Magic Chemical Co., 121 Crescent St., Brockton, Mass.

CEMENT-TREATED SUBBASE FOR CONCRETE PAVEMENT—Eleven-min film shows various methods used to construct cement-treated subbases. The color film describes "between-the-forms" construction, as practiced in California, as well as several variations of the more common "mixed-in-place" method of cement-treated subbase construction. The central plant mixed method is also described. The film is available on loan in the United States and Canada.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

REX CONCRETE SPREADERS—Bulletins and specification sheets are available on two new Rex concrete spreaders. Bulletin 60150 and specification sheet 60-S provide detailed information about Model MW, the manual widening spreader; Bulletin 60-152 and specification sheet 60-PFS cover details on the power and self-widening Model PW.—Chain Bent Co., Sales Promotion Department, Milwaukee 1, Wis.

A NEW CONCEPT IN ROTARY KILN DESIGN—Ten-page brochure illustrating and describing Vulcan's new concept in rotary kiln design.—Vulcan Iron Works, Inc., 730 S. Main St., Wilkes-Barre, Pa.

BRAB LIST OF PUBLICATIONS—List of current Building Research Advisory Board publications. Includes a brief description of each report.—Building Research Advisory Board, National Academy of Sciences, National Research Council, 2101 Constitution Avenue, Washington 25, D. C.

FORM-CRETE STEEL FORMS—General catalog on steel forms for precasting reinforced or prestressed concrete. Includes the latest in casting forms and developments designed to minimize stripping time, cleaning and resetting forms between castings.—Food Machinery and Chemical Corp., Form-Crete Department, P.O. Box 1718, Lakeland, Fla.

STADIUM AND AUDITORIUM CONCRETE (MBR-P-12)—Case histories of concreting problems encountered and successively solved in 16 outstanding stadium and auditorium projects in this country and abroad are reported in this 20-page publication. The role of Pozzolith as an aid in obtaining the concrete workability needed for this type of construction is cited.—The Master Builders Co., Cleveland 3, Ohio

TOOLS FOR THE TROWEL TRADES—Devoted almost exclusively to the tools used by contractors and journeymen in the trowel trades, the 72-page catalog shows the traditional tools and equipment and many new tools developed during the past few years. Contractor's equipment in all trades is also shown.—N. A. Goldblatt, Goldblatt Tool Co., 1910 Walnut St., Kansas City 41, Mo.

CENTRIFUGALLY CAST CONCRETE POLES (Bulletin No. 5901)—Eight-page engineering bulletin furnishes technical data on the design, manufacture, and erection of centrifugally cast reinforced concrete poles. Photos illustrate existing installations for substation structures, communication systems, street and area lighting, traffic signals, power transmission and distribution, and trolley bus and traction systems.—Centrifugally Cast Concrete Institute, P.O. Box 190, Thorold, Ont., Canada.

ELECTRON, X-RAY AND LIGHT MICROSCOPY MATERIALS RESEARCH—Brochure describes the type of service and equipment used for a wide variety of research and development projects at Ernest F. Fullam, Inc. The literature pictures and describes electron, x-ray, and light microscopes; explains electron and x-ray diffraction; x-ray spectrography; and electron probe x-ray microanalyzer. Fullam's up-to-date drafting room and machine shop designs and constructs or modifies specialized instruments of many types.—Ernst F. Fullam, Inc. P.O. Box 444, Schenectady, N.Y.

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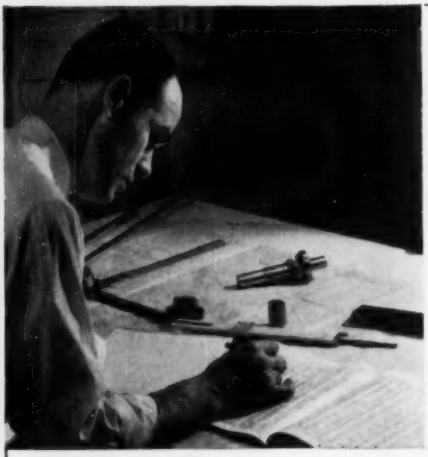

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